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# THE

# IMPROVEMENT OF RIVERS

## A TREATISE ON THE METHODS EMPLOYED FOR IMPROVING STREAMS FOR OPEN NAVIGATION, AND FOR NAVIGATION BY MEANS OF LOCKS AND DAMS

BY

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# PREFACE TO THE FIRST EDITION.

THE following treatise has been prepared in the hope that it may assist in meeting a want which has probably been felt by many engineers and others engaged on the improvement of rivers. With the exception of De Lagréné's "Cours de Navigation Intérieure," published in 1873, and which is now out of print, there is no work known to the authors which treats of this important subject except in a general way, or with sufficient detail for those engaged on actual construction or design. Much valuable information is to be found scattered through the various Government Reports relating to inland navigation, and matter of great interest has also appeared from time to time in various domestic and foreign publications. A search through these for information on any special point would, however, involve an expenditure of time and labor which few of those engaged in active work could give, even if the documents were accessible to them. Moreover, since the appearance of De Lagréné's work many new methods have come into use, and wider experience has been gained, and the authors believe that a treatise combining the results of theory and recent modern practice may prove of some utility in this field. They have accordingly endeavored to include all the important points of design and construction which are likely to be met with in ordinary practice, and the calculations have been simplified as far as possible so as to bring them within the range of those who do not possess a thorough technical education. It is hoped, therefore, that the book may rove of use not only to engineers, but also to inspectors, surveyors, and others who are engaged on the more practical side of work.

The authors desire to acknowledge their indebtedness to Brigadier-General G. L. Gillespie, Chief of Engineers, U. S. Army, and to the officers of the Corps of Engineers, for their courtesy in granting access to drawings and data connected with the works under their charge and permitting the publication of certain of them, and also to return thanks to certain civilian engineers for information on similar matters.

Their thanks are also due to Major W. M. Black, Corps of Engineers, author of "The United States' Public Works," and to Mr. Edward Wegmann, author of "The

Design and Construction of Dams," for permission to reproduce illustrations from those works; and to the American Society of Civil Engineers and the Association of Engineering Societies for similar courtesies; and to the Commission for the canalization of the Elbe and Moldau in Bohemia (M. W. Rubin, Chief Engineer of Construction) for information and photographs of that system of improvements.

JULY, 1902.

# PREFACE TO THE SECOND EDITION.

THE greater portion of this work has been rewritten and rearranged, and much new matter and many additional drawings have been added, especially in the portion treating of open-river navigation. The incompleteness of the first edition in this respect has, it is hoped, been largely remedied. The portion treating of canalization has been revised so as to include the latest practice, and matter in general, such as the surveying of watercourses, which can be found more fully described in other publications, or which can be obtained without difficulty from other sources, has been omitted.

The authors have drawn to a considerable extent on personal experience in describing matters pertaining to the execution and the results of work. Where they have been lacking in adequate knowledge, they have been fortunate in being able to supplement their labors with the help and advice of engineers more experienced than themselves, and they gratefully acknowledge therefore many additional courtesies and valuable assistance from the Officers of the Corps of Engineers, U. S. Army, and from many of their civilian assistants. Their thanks are due also, for valuable suggestions or assistance in other ways, to Sir William Willcocks, former Director-General of Reservoirs in Egypt; to Mr. Francis J. E. Spring, former Chief Engineer, Public Works' Department, India; and to engineers in Great Britain, France, Germany, Holland, Austria, Russia, and Australia.

Permission to use illustrations and subject matter from the Transactions of the American Society of Civil Engineers and from the Proceedings of the Institution of Civil Engineers is also acknowledged.

JULY, 1912.

# INTRODUCTORY.

In preparing this work for publication, especially after it had been subjected in a first edition to the test of practical use and to the criticisms and suggestions of engineers of wide experience, the difficulty became evident of treating the subject with satisfactory fullness from the standpoint both of theory and practice, within a compass of reasonable size. Of the theories of flow and the physics in general of waterways much literature was already available; of their practical application and of the results of experience there appeared to be but little. It has seemed best, therefore, in revising the book, to omit such matter as can be found in other works, or which is taught to students in their college days, and to state briefly the general principles of most importance, reserving the greater portion of the work for treating problems from the standpoint of actual practice, and of results accomplished or desired. Accordingly, for detailed information upon discharge, rainfall and run-off, methods of gauging, surveys, and similar matters, the reader must refer to some of the standard text-books on the particular subject.

It is believed that most engineers of waterways will agree that in this department of the profession a certain amount of practical experience is essential for successful work, more so perhaps than in any other. In the evolution of the modern bridge, for example, until the distribution of the strains had been worked out by theory, little real advance was made in practice, but with rivers the reverse has been the case; knowledge in regard to them is chiefly based on practice and observation, and through such means some theoretical principles have been deduced which nevertheless can only be applied generally and within wide limits. This is especially the case with rivers to be improved for open navigation. The combinations of slope, curvature, material of banks and bed, etc., found in them are so numberless that unless the engineer has dealt with somewhat similar conditions before, his prediction as to the effect of an improvement may be of uncertain value. Precise calculation of discharge and slope, or a similar theoretical determining of data, will not tell him what change a training wall or spur dike will produce upon a stream, nor where a dredge-cut ought to be located so as to produce and maintain the best results. Useful and necessary as such calculations may be, he must depend as much or more upon a close

observation of natural phenomena and on the training of his judgment so that he may trace correctly the numberless effects to their origin in the few great and universal laws governing the flow of streams, if he wishes his labors to attain success. The broader his comprehension of these laws and of their universalness of application, and the closer his observation of the analogies often supplied by Nature of the conditions he wishes to establish, the more certainly will he be able to join cause and effect. Thus a tree lying in a river and causing, through the impediment of its branches, a slackening of the flow, shows the effects of permeable dikes in creating deposits of silt; the eddies and other phenomena of spur dikes are often illustrated by the natural spurs of rock or clay to be found in most rivers; and the action of a stream flowing past a bank of earth interspersed with seams of loose rock, and gradually undermining them until the stone falls over and hinders further erosion, indicates how riprap can be used for bank protection. Similarly the broader applications can often be traced under varied forms. In the play of the currents and the building of sand-bars in the rivulet by the roadside, the same laws can be observed in miniature which mould the courses of rivers, and in an irrigation canal, fed by pumps from a sediment-bearing river, there can be watched the operation of the same causes which theory teaches have formed the flood-plains of rivers like the Mississippi or the Ganges. The muddy water, flowing into the straight artificial channel, commences gradually to silt it up at the upper end so as to gain the slope needed for the transportation of the sediment, and as the process goes on, bars, shoals, and islets begin to appear, the direction of the flow becomes diverted into bends and crossings, and in time the once straight channel will display, as far as the limits of the canal banks permit, all the characteristics of a sediment-bearing river.

From the observation of natural examples such as these many useful hints can be gained, and it should be borne in mind always that the more fully the engineer endeavors to understand and to work in harmony with the laws of Nature, the more probable will be his success.

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# THE IMPROVEMENT OF RIVERS.

## PART I

### *IMPROVEMENT BY REGULATION.*

### CHAPTER I.

#### CHARACTERISTICS OF RIVERS.

**Physical Features.**—The physical features of a country may be resolved into a series of inclined basins or valleys, each drained in its lowest portion by a stream, and each stream flowing toward some other watercourse or body of water. Each of these valleys is subdivided into smaller basins, and these again subdivided in the same manner, until the springs or rivulets at the source are reached. Thus the smaller valleys all lead to a great central valley through which flows a stream of greater or less magnitude, dependent upon the amount of rainfall and the character of the material. Under similar conditions the same laws apply to all these watercourses, regardless of size, and the smallest creek is characterized by sinuosities, eddies, bars, caving banks, and overflows exactly as is the greatest river.

The cooling of layers of air charged with moisture produces rain or snow, and the presence of forests, mountains, and valleys will also influence the temperature and precipitation. The quantity of rainfall in any one locality varies greatly from year to year, and is also much greater at some points than at others, usually diminishing as the distance from the sea or other place of great evaporation is increased. Thus in London, England, the mean annual rainfall is about 23 inches, while in certain districts in India, as shown by the records of the Royal Engineers, the rainfall has reached as much as 600 inches in one year.

Upon reaching the earth the rain-water is either taken up by the soil and vegetation, or runs off into the streams. That portion which runs off and immediately finds its way into the streams causes the freshets and floods. Its proportion will vary, like the portion infiltrated, with the character of the material upon which it falls and the

amount of vegetation present. Torelli has stated that a wooded mountain will retain four-fifths of an ordinary rainfall, and that the same mountain if denuded will retain but one-fifth, but this is disputed by others.* The portion of the rainfall which is held by the soil and given back as springs is valuable to navigation, since it replenishes the flow of rivers at times when there is no water running off the surface, and thus in regions containing considerable layers of permeable soils the summer flow is upheld by the natural storage of the winter and spring rains.

The amount of run-off will also vary considerably with the intensity of the precipitation, much more running off in what is called a hard rain, or thunder-shower, than in a gentle one. It will also be greater with a lasting rain, the earth then becoming saturated so that it can absorb no more. A frozen soil, or a soil hard-baked by the sun's rays, will turn off much more water than the same soil in its natural condition. The rains of summer, while usually heavy within a short period, are largely evaporated and absorbed by the dryness of the earth, but those of winter, falling on soil already saturated and possibly frozen hard, quickly find their way into the rivers. If these rains are warm and fall on snow, the conditions may become dangerous, as not only the rainfall but also the melting snow will find its way into the already overladen channel. Occasionally, however, the opposite effect has been known to result, and a heavy rainfall has been partially absorbed by a deep snow already on the ground and thus held from running off until the flood had begun to subside. A case of this character came under our observation, where a rainfall of 10 inches within 48 hours produced less immediate effect upon the river than half the amount would have done under ordinary conditions. The period of high water was prolonged, but its level was kept down much below that which experience had shown would ordinarily follow such a rainfall.

The melting of snows, when unaccompanied by rains, rarely produces floods in large streams, as the deepest snows when melted make but a few inches of water, and this reaches the rivers gradually. During the early spring the melting is reduced at night, or possibly stopped entirely, to be resumed again next day, but even if continuous, with the ground underneath frozen the quantity of run-off will not equal that of hard rains.

In many parts of the country the rivers are swollen beyond bounds during the spring and early summer months, only to run almost dry in the autumn, and in studying the characteristics of a stream with a view to its improvement, it is of course necessary to possess information regarding the rainfall, discharge, etc., and the data should extend over as long a period as possible.

**Bed.**—By the term bed is understood all that space ordinarily covered by water and lying between the lands on each side of a stream. In rivers which rise above the levels of these lands and overflow adjacent "flats" or "bottoms," the channel in

* For a discussion on the relation of forests to stream flow see paper by Lieut.-Col. H. M. Chittenden, Corps of Engineers, U.S.A., in the Transactions Am. Soc. C.E., for 1908. See also *Engineering News*, October, 1908, and other issues and articles about that date and later.

which the water is usually confined is called the minor bed, while the space occupied during flood-time is known as the major bed. The difference in width between the two is frequently very great, unless the major bed is confined by levees or embankments. The two sides of the minor bed are the banks, the right bank being always the one on the right hand of an observer looking downstream and the left bank the one on his left hand. In some streams they extend generally above flood-level, while in others the highest floods go over them. In exceptional cases they are submerged by ordinary high-water stages. On nearly all streams there are low banks at intervals, particularly at the mouths of the larger affluents.

The majority of rivers flow through alluvial lands, with varying strata of sand, clay, and gravel as a foundation, underlaid by rock. Here and there the hills approach on both sides, causing a narrowing of the bed, but the presence of hills close upon one side is nearly always accompanied by " bottoms " or level land upon the opposite side. In such situations, if the soil is easily eroded, the constant action of flowing water will result in a more or less continual changing of the banks and the bed. The amount of this erosion will vary with the character of the material and the transporting power of the water, which increases with the slope of the stream and with the mass of water in motion.

**Basin.**—The territory drained by a river is called its basin. It therefore reaches to the mountain tops and includes the valleys of the tributaries as well as that of the principal stream. Its highest point is the watershed which divides it from another basin, while its lowest point is the bed just described. Its influence on the river is governed by the character of the strata of which it is formed. When these are permeable, the rain sinks into the earth and reaches the bed gradually; when they are impermeable, the water finds its way to the river rapidly, and the river will therefore fluctuate slowly or rapidly as its basin is permeable or impermeable.

**Transportation of Sediment.**—The mountain sides and valleys are slowly but surely being carried to the sea by the rivers which penetrate them. In the upper portions of these, if the slope is great and the volume of flow sufficient, the material transported is coarse and heavy; as the descent of the valley proceeds this material becomes finer, until near the mouths of most rivers it consists of fine silt. The depositing of this material leads to most of the difficulties encountered in river navigation. Evidences of it may be seen on the banks after every freshet, or if a sample of the water be allowed to settle, there will be found at the bottom of the vessel a small quantity of sediment which may be separated into two general classes, one of very fine particles of mud or clay, and one of sand of varying grades of fineness. The materials of the former class, being light, are easily kept in motion, while those of the latter have a constant tendency toward the bottom, and will be precipitated in places where the velocity is reduced.

In making observations of the amount of sediment, it is necessary to secure

amples of water from various depths and at different points of the section. For this purpose a tin can attached to a graduated pole and having valves at the top and bottom opened by cords or wires leading above water may be used. For great depths it is necessary to weight the can and use a cord instead of a rod. The samples are allowed to settle and the sediment in each is then weighed, or measured; a proportion can thus be established between the volume of water and the matter transported.

Numerous experiments have been made to determine the velocity at which the current begins to move various materials, but the differences in local conditions render the information obtained of uncertain value. One of the chief of these is the volume of discharge. A river with a fall of one foot per mile can transport a large amount of heavy sediment, whilst a brook with a similar fall can hardly carry silt. The experiments of Du Buat gave the following ratios between materials and velocities:

| | | | |
|---|---|---|---|
| Potter's clay | 0.26 | feet | per second |
| Sand deposited by clay | 0.54 | " | " |
| Large angular sand | 0.71 | " | " |
| Gravel, size of peas | 0.53 | " | " |
| Gravel, size of beans | 1.07 | " | " |
| Round pebbles, as large as thumb | 2.13 | " | " |
| Angular flint stone of size of hens' eggs | 3.20 | " | " |

The following, however, were found to be the velocities under which movements of gravel occurred on the Loire:

| | | | | | | |
|---|---|---|---|---|---|---|
| Gravel, | 0.04 | inch | in diameter | 1.64 | feet | per second |
| " | 0.16 | " | " | 3.28 | " | " |
| " | 0.39 | " | " | 4.92 | " | " |
| " | 0.67 | " | " | 6.56 | " | " |

The velocities above quoted—certainly those of Du Buat—are presumably the ones under which materials already in motion would continue to be carried along. From observation of actual conditions in rivers one is led to believe that considerably higher velocities would be needed to cause much erosion where the particles were uniform in character. Thus in the irrigation canals leading from the Nile, which are excavated through alluvial earth, it was found that when the velocity fell to about 2 feet per second, or less, the silt in suspension began to cause deposits, while at 2.3 feet per second there was no deposit. An increase to 4 or 5 feet developed tendencies to erosion, and the rate of 3¼ feet per second was finally adopted wherever practicable, as it avoided both deposit and scour. On certain of the irrigation canals in India this rate is from 1½ to 2 feet per second.

Buckley states that material in place will usually resist the following velocities per second: Sandy soil, 1 to 2½ feet; ordinary clay, 3 feet; compact clay, 5 to 6 feet; gravel and pebbles, 5 to 6 feet.

The following data in connection with sediment in the Arkansas River are of interest.*

"The matter in suspension is greatest during a sudden high rise; but after the water in the stream stands at any high mark for a few days, the decrease of the amount of suspended matter it carries is very marked. The amount of sediment carried by the river varies widely also with the same gauge-reading at any stage, being greater with a rising and less with a falling river.

"The greatest amount of sediment found in the water during the year under consideration was 225 grains to the gallon ($\frac{1}{259}$) when the river stood at 17 feet on the gauge, and after protracted rains. (Extreme high water at Little Rock is about 28 feet.) It should be added, however, that while this high water may be taken as a type of the ordinary rises, there are times when there is little or no rise, and no increase in the volume of water discharged, but a very marked increase in the amount of mechanically suspended matter. In October, 1891, occurred one of these so-called 'red-rises' of the Arkansas River, and although the river was quite low—marking only 3.9 feet on the gauge—it carried 761 grains of matter to the gallon, of which only 46 grains was matter in solution (that is, the matter in suspension was $\frac{1}{82}$ by weight).

"The matter in solution bears no constant relation to the volume of water, though in a very general way it varies inversely with the volume of the water, and ranges from 11 to 70 grains to the United States gallon. The amount carried down, in this form, from October, 1887, to September, 1888, was 6,828,350 tons. During the single month of May, 1888, 1,161,160 tons were carried out in solution. Taking the observations for the entire year under consideration, the matter in solution is equal to about 0.31 of that in suspension. These relations, however, are not constant. In November, 1887, for example, the dissolved matter was more than six times as much as the suspended matter—while on October 13, 1891, the suspended matter was more than thirteen times the matter in solution."

Some idea of the enormous forces at the disposal of a river for the lifting and transportation of sediment and for the various related functions may be obtained from the Rhine, which, with an average annual discharge of 70,000 cubic feet per second between Bingen and the Dutch frontier, generates and spends every minute between those limits the equivalent of two million horse-power.

**Relation of Sediment to Cross-section and Slope.**—The principles involved in the transportation of sediment compose the chief law governing the regime of rivers. The action of the rainfall and of the flow on the various materials composing the banks

* "Observations upon the Erosion in the Hydrographic Basin of the Arkansas River," S. C. Branner. See also p. 49.

and bed of the main river and its tributaries erodes the soil to a greater or less extent, and where the materials are alluvial, the water becomes charged with a burden of sediment which fluctuates from mile to mile, but which always tends to remain in exact proportion to the power of the water to carry it.. Where the local swirls or the velocity of the current are just strong enough to keep the sediment from settling, no change occurs, but when the motion becomes less, some of the burden is immediately let fall. If on the other hand stronger swirls or swifter velocities commence, the surplus forces eat into the banks and bed and take up a weight of new matter which always tends to balance exactly the added power. This process, simple though it may appear, regulates the configuration of every rivulet and river flowing through alluvial soil.

These phenomena may be illustrated by introducing slowly a quantity of fine sand into a rivulet of clear water and moderate fall. The amount may be regulated by using a box with a slide in the bottom and allowing the material to flow out in a steady stream. A gradual change will then take place in the rivulet; where the current runs slowly bars will begin to appear and backwaters will silt up; the pools will become shallower and the bed will rise; and there will be found a tendency towards a more even slope. The sediment will collect at first near the box, and when it has forced the water to create a new slope steep enough to wash the particles downstream—the inflow of sediment and the volume of water remaining constant—these will begin to gather further down, and the slope, while retaining the same general angle of inclination, will gradually increase in length until it reaches some hole or sudden slope where the water can finally let go or carry along its burden. If now the amount of sediment is increased, similar results will occur; the slope at the box will steepen and all the bed will rise until the velocity has again become great enough to carry down the load.* If the sediment is cut off the water will at once attack its bed, beginning as before at the head, and will continue to erode and carry away the material until it has so far reduced its slope—and therefore its velocity—that the balance of the forces is again restored and erosion stops. By changing the volume of the water irrespective of that of the sediment, and by varying the quality of the latter, the phenomena of shifting of channels and other vagaries of alluvial rivers will become manifest, and will afford some idea of the numberless variations possible to a sediment-bearing stream.

If no hole or sudden slope is found where the water can dispose of the material, and no other rivulet is met with which will carry it away, the bed will continue to rise indefinitely. Such cases, however, do not occur in Nature; the nearest parallel is where a river discharges into a body of still water, as a lake or a sea, in which case the bed is gradually raised from the mouth as far upstream as the increase of deposit can affect the slope. This follows from the law of transportation of sediment,

* An example of the effects upon a river of excess of sediment is given on p. 33.

before described; the material let fall at the mouth gradually builds up the outlet until it lessens the slope of the flow above. The velocity thus decreases; part of the suspended matter in transport is let fall, and this raises the bed until the slope needed to carry the burden is again restored. The process of change is continuous, but it is so slow as to be practically imperceptible. The Nile offers an interesting example. On Roda Island, opposite Cairo, there stands an ancient gauge whose records have been preserved since A. D. 861, and these show that the stage of water necessary at the present day to secure flood irrigation everywhere is four feet above what was needed then, indicating that the bed of the river and the surface of the irrigated lands have risen at the rate of 0.39 ft. per century. Similar records show that the level of the highest floods at Assuan, 560 miles upstream from Cairo and below the last of the cataracts, has risen 6.9 feet in some 1600 years. This gives a rate of rise of 0.43 ft. per century. This record is based on a maximum flood mark made by an officer of the Roman garrison in the reign of the Emperor Severus, and was compared by Napoleon's savants with the levels of maximum floods occurring during recent times, the difference being as just stated.*

According to the foregoing there should be in every part of a river a combined proportion between the discharge, the velocity, and the cross-section of the bed, or the amount of erosion effected by the stream. As a flood rises or falls it should fill up or scour out the varying portions of the bed to the exact amount needed for the passage of the water and its burden; the energy spent should balance perfectly the work done. While the tendency to do this is always in evidence, cases where it is entirely fulfilled are rarely, if ever, found. Apart from the local disturbances due to spurs of rock or clay, or to other factors which set up unusual reactions which the river cannot overcome, the numberless variations in the rapidity of rise and fall, the time elapsing between the floods, the effects of local rains, etc., all have their influence in preventing the stream from attaining the final equilibrium throughout its length. Measurements in low water show, however, that where a considerable time has elapsed after a flood, the bends, where the water runs slowly in the low season, tend to silt up in proportion as the shoals, where the water runs fast, tend to erode. Examples have been found in Indian rivers where bend channels have filled during low water with nearly 40 feet of silt. Measurements taken on the Brazos River, an alluvial stream of Texas, over a distance of a few miles, and comprising several bends and sand shoals, showed that the comparative areas of the bend and of the shoal sections, where conditions were normal, had become equal within about 10 per cent. The soundings were taken after a long period of low water, when there was still a fair discharge, but no sediment in suspension, and when the erosion of the shoals had practically ceased. The slope of the latter had been scoured down until the velocity of the current over them was nearly equal to the average of that in the bends, and the cross-sections at the

* "The Nile in 1904." Sir William Willcocks.

bends and shoals varied within the comparatively narrow limits stated. Similar equalization of areas was found in the Merwede, one of the large outlets of the Rhine.*

With rivers flowing through rocky beds the variation is naturally much greater, owing to the hard material. On the upper Tennessee River the cross-section of a pool in low water was found in many cases to be ten times as great as the cross-section at the neighboring shoal, even though the distance apart was only 500 feet.

**Slope.**—The slope of a valley determines the velocity of the stream. When it is gentle very little erosion takes place, unless the soil is unusually unstable, but where the slope is great the water attacks the soil, and the river-bed takes a form dependent upon the quantity of water and its velocity. As the river rises, its slope and velocity increase, and it is not possible for it in many cases to preserve a fixed bed or banks. From this it follows that its profile does not remain constant and uniform for all stages and under varied conditions, nor does the cross-section. The slope also varies with the local conditions of the channel, which in some rivers are liable to

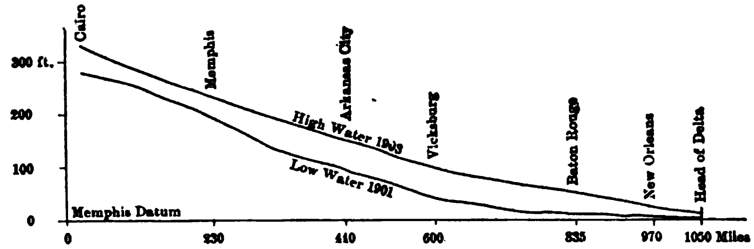



FIG. 1.—High and Low Water Profile of the Lower Mississippi River.

constant change, so that it may not be the same for any length of time on a given length of river, and it is, therefore, by no means uniform throughout the course, nor does it decrease with any fixed regularity as the river approaches its mouth. A general profile, however, would show that the greatest slope is near the source, decreasing more or less regularly as the river follows its course. (See Fig. 1.) De Mas draws attention to the fact that tributaries have an important influence on the local slopes, and that below the confluence of a tributary bearing much sediment, the bed of the main river usually becomes steeper, while the slope above is reduced. Such an affluent brings in an amount of material which the principal stream, having its own burden to carry, cannot entirely absorb. As the result, deposits occur and the bed of the river is raised until equilibrium is again reached between the velocity and the sediment. On the other hand, if the main river carries much sediment and the affluent little, the balance is disturbed in the opposite way, and the waters after mingling commence to gather up material from the bed until the river has again its full burden, the excavation being greatest just below the tributary, and decreasing downstream. This results in the slope below being decreased, and becoming less than

* International Congress of Navigation, 1894.

that above the confluence. According to this, there should be found corresponding differences of slope at the junctions of the Mississippi, the Missouri, and the Ohio rivers. The upper Mississippi, compared with the Missouri, is comparatively clear, while the volume of the sediment of the latter is enormous. The former river, before joining its muddy tributary, should have the lesser slope; after the two streams have mingled the proportion of sediment to the volume of water is reduced for the one and increased for the other; the slope should therefore become a mean proportioned to the sediments and volumes, and should become less than that of the Missouri, and greater than that of the Mississippi, before their confluence. As the combined flow of the two rivers approaches the Ohio, the sediment becomes reduced by grinding, and the slope should become less. When the Ohio is reached the former conditions are reversed; the tributary is somewhat the clearer, and the main river somewhat the more muddy. Above their confluence, therefore, the slope of the Ohio should be the less and the slope of the Mississippi the greater; after their junction the proportion of sediment to volume is lessened for the Mississippi and increased for the Ohio; the resulting slope should therefore be less than that of the Mississippi and greater than that of the Ohio before their junction. Such conditions actually exist. The slope of the Mississippi in the last 43 miles above the mouth of the Missouri is 0.48 ft. per mile; that of the Missouri for its final 50 miles is about 0.9 ft. per mile. Below their junction the slope becomes 0.81 ft. per mile for the succeeding 18 miles. For 48 miles above the mouth of the Ohio the slope of the Mississippi is 0.64 ft. per mile while that of the Ohio itself is probably 0.2 or 0.3 ft. per mile. For the 50 miles below their confluence the slope (in floods) becomes about 0.4 ft. per mile. A similar example is offered by the Rhone, which at its meeting with the torrential Isère has a mean slope in a distance of 54 miles of 2.8 feet per mile above the mouth, and 4.1 feet below. At its meeting with the Saône, however, which is of less torrential regimen than the Rhone, the mean slope in a distance of 65 miles is 4.3 feet per mile above the mouth, and 2.6 feet below. Similar phenomena are found on the Loire, the Po, and other rivers.* The effects of the first condition can often be traced in many streams in the low banks above the confluence and the broken channels and the islands found below. The river has been unable to carry all the sediment, and has disposed of it by building it into islands, as well as by depositing it along its bed.

To the same cause—the deposition of sediment on reaching clearer or more tranquil waters—is due the formation of the islands often found at the mouths of rivers. The material in settling builds up sandbanks which grow into islands, and these eventually may cause the river to divide into many secondary branches, such as are seen at the mouths of the Rhine, the Amazon, or the Ganges.

The slope of a river, except with those prone to rapid changings, is usually an index of the material of the bed, and if the character of the latter is known, the tendency

* "Rivières à courant libre," p. 53.

to erosion can be closely estimated. General Rundall, R.E., found the following slopes per mile on rivers in India:*

For beds entirely of boulders, 15 to 30 feet.
Beds of solid rock, or rock mixed with hard soil, 5 to 6 feet.
Beds of hard uniform clay, 5 to 6 feet.
Beds of coarse sand with gravel, 2 to 3 feet.
Beds of coarse sand only, 1 to 2 feet.
Beds of fine sand, 4 to 12 inches, with unstable banks.

In America the upper 30 miles of the Ohio with a gravel bed have a s'ope of about $1\frac{1}{2}$ feet per mile, while the middle and lower portions, where the gravel begins

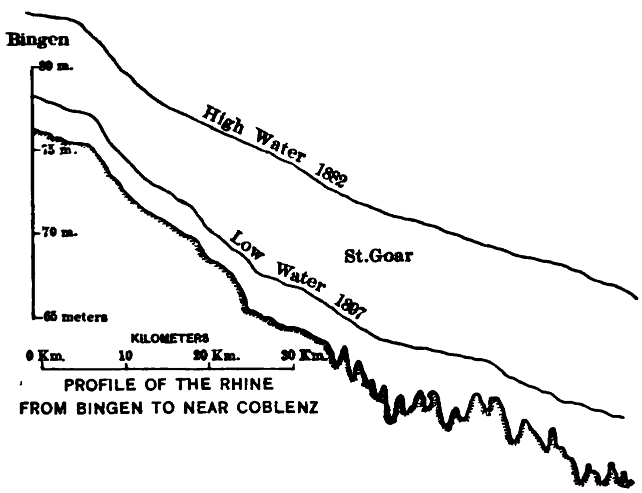



FIG. 1*a*.

to disappear and to be replaced by sand, have slopes from 2 or 3 inches to 6 inches per mile. The lower portion of the Mississippi, with a bed of fine sand and silt, has slopes from $1\frac{1}{4}$ to 3 inches per mile.

In low water the profile of the water surface consists of a series of approximately level pools united by a greater or less fall at each shoal. A moderate rise obliterates these falls, as the water usually rises more quickly below a fall or a dam than above, since it must gain hydraulic head to carry it through the pool to the next fall below. The surface of the river then becomes (Fig. 1*a*) approximately uniform with lines joining the crests of these shoals (neglecting for the moment the effect of the slope of the flood-wave), and if the rise continues, the profile becomes approximately coincident with the elevations of the most pronounced shoals, modified of course by the local variations in width of bed, etc.

* Proceedings, Inst. C.E., vol. lx.

The following table shows the distances from the sea at which the elevation of 100 meters (328 feet) is attained in certain rivers, and affords a good comparison of the differences of slope:*

| | | |
|---|---|---|
| Rhône | 134 miles | (at Valence) |
| Garonne | 224 " | (near Toulouse) |
| Loire | 248 " | (near Orleans) |
| Weser | 249 " | (near Karlshafen) |
| Oder | 326 " | (near Breslau) |
| Seine | 346 " | (near Marcilly) |
| Rhine | 386 " | (at Carlsruhe) |
| Elbe | 412 " | (near Dresden) |
| Volga | 1243 " | (near Nijni-Novgorod) |

Additional data regarding the slope of various rivers will be found at the end of this chapter.

**Formation of Bed.**—The following description by M. Janicki illustrates the development and effects of the forces causing erosion:† "Let us examine how the bends and the bars are formed. To better understand the details of the process, let us suppose a plain, absolutely regular, and not horizontal, but slightly inclined in a certain direction. Let us suppose, further, that in the direction of this inclination a canal is dug having a certain cross-section, a regular bottom, a slope parallel to that of the ground, and regularly constructed side-slopes. Into this canal we will admit a river at its maximum discharge. This river has a certain velocity of current which undermines the banks and cuts out the bottom of the canal, and we shall accordingly witness the following phenomena: the water begins by detaching a few small particles from the bed and cutting away the two banks equally, but as in nature no ground is absolutely homogeneous and of the same tenacity throughout, it finally happens that at some point one bank yields sooner than the other. This first undermining gives rise to a slip or mound which destroys the symmetry of the original profile, and deflects the current toward the opposite shore. Soon this second shore, in its turn, crumbles away just where the deflected current strikes it most powerfully. This new mass of fallen earth cannot remain at the foot of the bank whence it came. The current here being already increased, it is carried a little lower down stream and there forms a bar, which in turn directs the current toward the bank whence the trouble first started. If we add to this that the current once turned aside from the original straight line wanders farther and farther away by the very force of inertia, and that this deviation continues until, from the very fact of its digression, the slope and velocity of the current are reduced and come into equilibrium with the resistance

* "Rivières à courant libre," p. 55.

† "Notes on The Navigability of Rivers," 1879.

of the banks, we shall see how bends are slowly and gradually formed, making detours both to right and left of the line of maximum slope. The elongation of bends only ceases when the slope is so diminished from the increased length given to the course through which the water must flow that there is no further tendency to produce scour of bottom or banks. Theory and experience teach us also that velocity of current is determined not only by the inclination of water-surface, but also by the form of the bed; that is, by the form of its cross-section. For any given soil and any given inclination there is but a single form of flowing cross-section which will give the maximum velocity of current with the least resistance. In the above assumed case of an artificial river whose curves are freely developed, the form of cross-section will undoubtedly vary according as we consider the bed at the head of a bend, at its apex, or at the point of passing from one bend to another; these variations, however, will be constant at similar points, for the slope, by reason of the increased length, has become almost uniform, and the other two factors—velocity of current and nature of the bottom—being likewise uniform, the depths of the sections and their widths will have a constant maximum limit. But what will happen if one of the banks is higher or more solid than the other, and if from this, or any other special condition of the surface of the adjacent bottom land, the river cannot sufficiently increase the length of one or more of its bends? It cannot maintain a very steep slope; the nature of the soil forbids that. In such a case it is evident that the stream will more and more scour out its bed, and deposit the débris in those places favored by the topographical features of the valley; that is, where it is possible to build up the bottom. In other words, the result will naturally be an elevation of the bed of the stream, and this elevation will act as a cross-dike, damming the river like a weir; it will withstand and considerably diminish the force of the current; the water will flow over it in a thinner sheet, and, following the exterior slope of the bar, will fall into the lower pool by a route much shorter than through a bend, and without a tendency to cut away its bed; for we know that with a given slope the velocity of bottom flow diminishes with the depth of the sheet of water.

"If the discharge of our artificial river always remained the same, at the end of a certain time it would finally come into equilibrium with the resistance of the soil throughout its whole length. The bends and the bottom, when they had once assumed their proper form, would retain it, whatever might be the consistency of the soil. But the discharge of rivers often varies through wide limits. Into our artificial canal we let loose a river at flood-height. If the discharge should gradually diminish, the water-level would fall. At deep places the section would always be sufficient for the passage of the water in spite of any diminution of the slope caused by lowering the level, and the regimen of those parts would not vary; but wherever the bottom had previously been raised these elevations would act more and more as if they were dams that closed the whole width of the river. The water would flow over them, would attack them,

and dig out a new low-water channel which would have a width and a depth adapted according to hydraulic principles, to the nature of the obstructions, to the fall (from the upper level to the lower), and to the low-water discharge.

"When the water again rose, the swell, starting at points above, with a current increased with the slope, would bring down new material and fill up the low-water channel already formed, and thus reconstruct the bar to its former height. This work of lowering and reconstructing bars is repeated at each freshet. The longitudinal profile of a river taken during low water shows that it is composed of a succession of pools where the fall is generally less than the mean fall, and also that the pools are separated from each other by bars where the fall is greater than the mean fall of the river.

"We have taken a purely theoretical river for an example, and we have supposed that in the beginning it had a regular bed situated in a plain where the soil was homogeneous. If we now take into account the diverse topographical features, and the geological complexity which ordinary river valleys present, we shall have that variety of cross-sections, of surface slopes, and of more or less pronounced bends which are found in free rivers.

"Our attention is thus called to the intimate relationship that exists between all those phenomena which at first view appear so entirely distinct from each other. No one can anywhere interfere with the curvature of a river, its slope, or the depth of its cross-section, without immediately causing, either above or below the point, some change in the pre-existing conditions of its equilibrium. This equilibrium, we must not forget, is not a static, but a dynamic equilibrium, and, therefore, in the present condition of the science it is very difficult to determine its exact conditions in advance.

"We have shown that the want of solidity in the soil is the natural regulator of the rapidity of the current. This lack of solidity, consequently, leads to the formation of bends and bars, which re-establish the equilibrium between bed resistance and velocity of current.

"The status, the general character of a river, therefore, depends on the united action of these three factors: the discharge, which is variable; the magnitude of the slope, which is likewise variable, and the nature of the soil, which is variable in different localities.

"For a river to be navigable it is necessary that it should have a sufficiently deep channel throughout its entire length. A river may have much water, but if the fall is considerable and the soil unstable it cannot have a deep channel. On the other hand, there are rivers with a relatively small discharge and great fall which are yet quite suitable for navigation owing to their hard bottom."

**Typical Features.**—*Shoals, Bends, and Crossings.*—The accompanying sketches (Figs. 2 and 3) illustrate the typical features of an alluvial river. It will be seen that the bed consists of a series of bends closely resembling transition or compound curves united by straight reaches, and that the main volume of the flow, under the influence

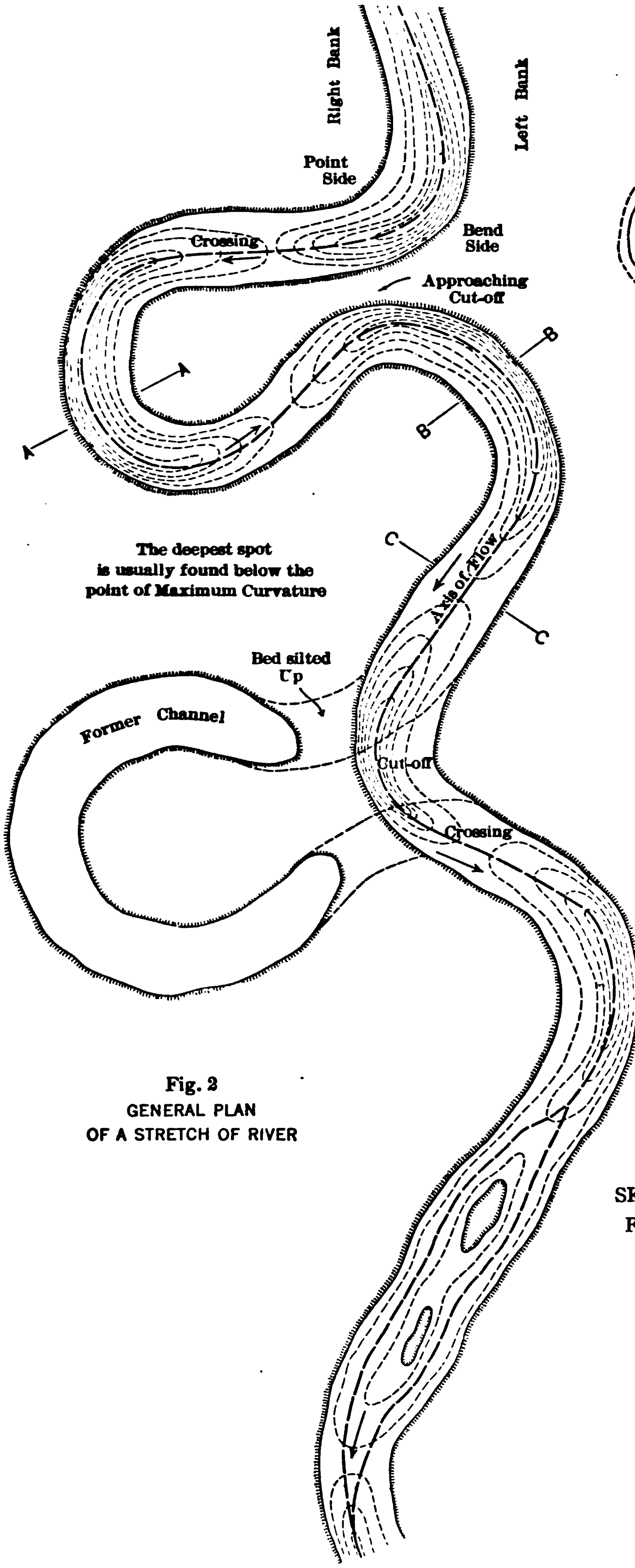



Fig. 2
GENERAL PLAN
OF A STRETCH OF RIVER

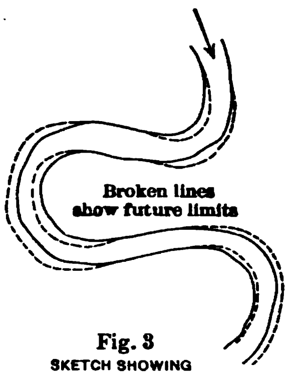
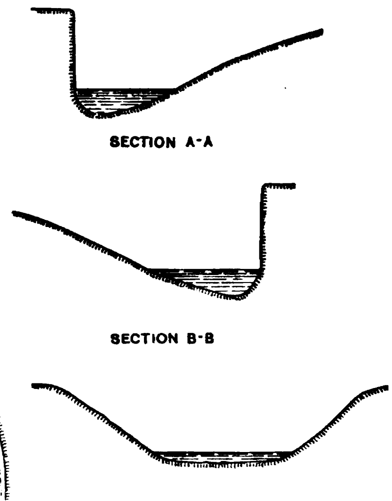



Fig. 3
SKETCH SHOWING
TENDENCY OF A RIVER
TO SHIFT ITS BED




SKETCHES SHOWING THE TYPICAL
FEATURES OF AN ALLUVIAL RIVER.

of the centrifugal forces, follows the concave shore, and not the center of the river, until a straight portion is reached, when it crosses over gradually to the commencement of the next bend (these portions of the stream being therefore known in America as "crossings") and follows along its concave side until the next crossing is reached, when it passes to the other side as before. Along the concave bank therefore is found the deep water, with the deepest spot usually below the point of sharpest curvature; as the flow approaches the crossing, it spreads out, and there being naturally in that portion a wider cross-section and an absence of the eddies and reactions which occur in its passage along the curve, and which prevent the sediment from settling, it will begin to deposit its burden. The wider the bed at a crossing, the greater will be the slackening of the current during a flood, and the more sediment will be let fall. As the result, at these crossings are always found at low water shallow and uncertain depths, which in the widest places often build up into bars and islands. At the first slackening of the flow the heavier particles are deposited, while the lighter ones are carried along until the velocity is reduced still more further on. It is well known to dredgemen that the coarsest sand or gravel is to be found at the head of a bar, and the finest at its foot. These deposits occur of course during floods when the water is charged with sediment, and result in the low-water slope of the stream consisting of a series of long and short slopes, the short ones being the steeper and occurring at the shoals.

Although the bed between the bends generally widens and is accompanied by shoals and more or less uncertainty of flow, yet at such points there are occasionally found deep and stable channels between narrow banks, and which retain their cross-section from year to year with little change. These usually exist in straight or nearly straight reaches with the bends above and below of long radii, where the current flows nearly parallel with the banks, and hence little erosion takes place with its accompanying shifting of the bed. To quote from a government report on the Mississippi: "At many points along this river are found reaches of moderate length, where the width is too small to allow the formation of a middle bar. At these places, often very narrow, the whole volume of the water, even in floods, flows in a concentrated mass, the direction of the current being parallel with the banks. From the date of the earliest land surveys (1817) these places seem to have remained almost unchanged, there having been little erosion. As the river rises, the bed gradually deepens, and it fills up again as the river falls, leaving the relative section unchanged from year to year, while the depth even at low water is much greater than the average."

*Influence of Curvature.*—The general course of a river is thus composed of a series of curves and tangents, the curves, when not disorganized by hard local strata, being of the transition type—that is, they commence at the point of tangent with a radius which decreases until the apex is reached, when it increases until it meets the following tangent. The flow is governed by the same laws of restricted motion as those which have been found to exist with fast-moving trains upon a railway. If the

alignment of the latter consists of tangents joined by plain arcs of circles, the track tends to become displaced until it has assumed the outline of a transition curve, showing that the change of motion should be gradual and should increase and decrease gradually to accord with natural law.

The curves of a river are of all varieties, each stream appearing to have minimum and maximum limits within which it can maintain a stable channel. If the former is exceeded, erosion is set up which increases the curvature; if the latter, bars and shallows form which modify the channel and reduce it to smaller curves. The general alignment may be separated into two typical parts, the first being where a curve in one direction is succeeded by a crossing or tangent and then by a curve in the opposite direction, and the second where the tangent lies between two curves in the same direction, that is, both trending to the right or to the left.* The latter class is rare, and in many cases an examination of the currents in the tangent will show that the reach is in reality only a curve of large radius, or that the flow is subdivided into short reversed curves of the first class and does not travel along a straight line. It seems to be a universal tendency of flow that the movement should progress with a constant swinging to and fro, right-hand and left-hand curves succeeding each other with hardly an interruption, and if a stream of water is made to run over a smooth expanse of sand, without direction being given it, it will eventually cut out a channel where curves of the second class are rare, and the general alignment is of the first class, of reversed curves and crossings.† This principle, termed by French engineers the oscillation of flow, is one of the most important of a river's elements, for by it definite tendencies are imparted to the current without which the regulation of complicated channels would be little else than guesswork. It is supposed to be dependent on the principle of economy of natural forces, manifested in the tendency of Nature to accomplish her results with least labor. The chief function of a river being to discharge the flow with the accompanying burden of sediment, it should strive, in accordance with this law, to create that cross-section of channel which will permit it to fulfill its labors with the least waste of effort. The chief requisites of such a channel would be stability and depth, since a shifting course leads to constant and useless toil in the erosion and building of sand-bars, and a shallow one gives rise to a friction from which a deeper channel would be largely free. These two requisites appear to be secured by a river more easily with motion in a curve than along a straight line. If a river were admitted into a straight, wide valley with a rock surface sloping towards the outlet and slightly hollowed in the transverse direction, it would of course take the shortest route to the mouth, and would be unable to lengthen into bends. The controlling force would be that of gravity alone. If, however, the valley were of alluvial soil instead of rock, erosion with its complications would at once begin, and the river would be in the grasp of two contending influences, the one, that of gravity tending to drive it down the shortest route, and the other that

* See Pl. 1*a*. † See experiments by M. Fargue and others.

derived from the necessity of its keeping as far as possible an equilibrium between the velocity of flow and the nature of the bed. The more this equilibrium can be preserved the less will be the labor of a river in carrying out its work, because the erosion will be less. The stream therefore lengthens its course in order to reduce its slope and therefore its speed, and then appears to come into play the far-reaching principle which guides it in moulding the type of channel which will best facilitate its labors, and allow it to transport both flow and sediment with the least waste of energy.

The advocates of this theory claim that upon this elemental law rests the fact that alluvial rivers rarely flow, or can be made to flow, in a straight course without deterioration of channel. When moving in curves, the centrifugal forces cause eddies which prevent the sediment from settling and so impeding the flow and creating a shallow and uneconomical cross-section; and the river holds to the curve as long as it can, and until it is forced by the natural conditions to pass into a crossing and commence a curve in the opposite direction. The crossing, the straight portion, is the danger point; there the currents begin to lose their unity of action; the eddies slacken, and the flow becomes more tranquil; and thus the sediment gradually settles on the bottom and creates a barrier which further lessens the power of the river to preserve its economy of channel. When the water has struggled past its difficulties, it reunites at the commencement of the following bend, and the cross-section again becomes deep and suitable to ease of flow. In an alluvial river there is, of course, a constant change in progress; the bends are undergoing erosion and occasional cut-offs disorganize conditions, but the influences which control it remain unchanged, and the stream will maintain its tendency to flow in curves, because with them it appears more able to maintain a cross-section which will allow it to carry the discharge.

As might be expected, the course of a river is the most sinuous where the banks are of the softest material, since there its course has to be lengthened or the banks would be destroyed, and at the same time it has to preserve a deep and easily-flowing channel in order to carry off the products of the erosion. How far each of these two influences affects the resulting conditions of curvature it is impossible to say, but the principle requiring ease of flow is undoubtedly far-reaching. Thus even in the sluggish portions of rivers the tendency to swing from side to side is still apparent, although the increase of distance caused thereby is too slight to have much influence on the erosion of the adjacent banks. The course of the Mississippi for the last seventy or eighty miles below New Orleans consists almost entirely of a series of reversed curves of radii so large that they lengthen very little the course of the stream, and although the banks and bed are of fine silt, practically no erosion takes place. On the portion beginning at the head of the delta and extending upstream for a distance of fifty-three miles measured along a straight line, the length along the line of the channel is about $58\frac{1}{4}$ miles, an increase of only 10 per cent. The flood slope of this portion is about 0.10 foot per mile, so that a reduction to a straight line would mean an increase

in the slope to about 0.11 foot per mile. The stream, which flows through boundaries created by itself and of the lightest materials, could apparently have worked out for itself a straight course without endangering its banks, since the dividing line between stability and erosion does not lie within such slight variations of speed as this difference in slope would make. The burden of sediment, however, is enormous, and it seems therefore probable that the curves are necessary in order to preserve the ease of flow and to produce the swirls and the restlessness in the water which best seem able to prevent sediment from settling. (See also Fig. 27.)

*Movement of the Filaments of Water.*—An examination of the surface flow of a river will show that the filaments are moving generally parallel to the banks. The presence of bends, islands, etc., varies this movement locally, but the general tendency remains unchanged. Beneath the surface, however, and in proportion to the depth, the direction of the filaments undergoes a constant change, not only from the vertical eddies and movements described further on, but also from the varying contours of the bed which cause lateral movements as the water shoals or deepens. This is illustrated in Fig. 3*a*, which shows that as the flow approaches a shoal and the depth under any one line of filaments becomes less, the water is pushed over towards the concave section, which accordingly becomes the deeper in proportion as the convex or shoal portion becomes shallower. At the other end of the shoal, where the crossing begins, a reverse action takes place; the bend becomes shallower and the filaments are pushed away from it and spread again over the bed until another shoal or bar is met with which pushes them back to the deep channel. This action of the flow, which received the first careful investigation by M. N. de Lelavski, a Russian engineer, has been termed by European engineers "l'appel des eaux," or "the attraction of waters," signifying that the deeper channels tend to attract the filaments from the shallower ones, and to concentrate in this way the flow.

*Scour.*—In low stages, when the river has little more sediment to deposit, it begins to find that the material piled up in these crossings has raised the bed and created temporary dams there, causing an increase of slope which the river utilizes in attempting to scour the obstructions away. This, however, it can only do to a certain degree, since investigations show that once a particle of sediment has been dropped, a much higher velocity is needed to pick it up again than was required merely to transport it. Hence, if the higher velocity of the flood was unable to carry it, the velocity of the low water will be unable to disturb it unless the temporary fall created by the deposit itself sufficiently increases the speed. With gravel shoals the water has to increase its speed until it can roll the pebbles bodily down the slope; under favorable conditions their clicking can be distinctly heard. With sand shoals the process is more elaborate; the friction of the flow over the bottom sets up a series of small eddies transverse to the current, and these in turn throw up sand ripples or ridges perhaps an inch or two in height, with a short downstream slope and a long upstream one against which

the water beats, disturbing the particles and carrying them downstream. The effects are the same as those produced by the wind blowing over a level of fine dry sand, and the outline of the ripples is similar, as it is similar to the outline of ripples upon the water when driven before the wind.

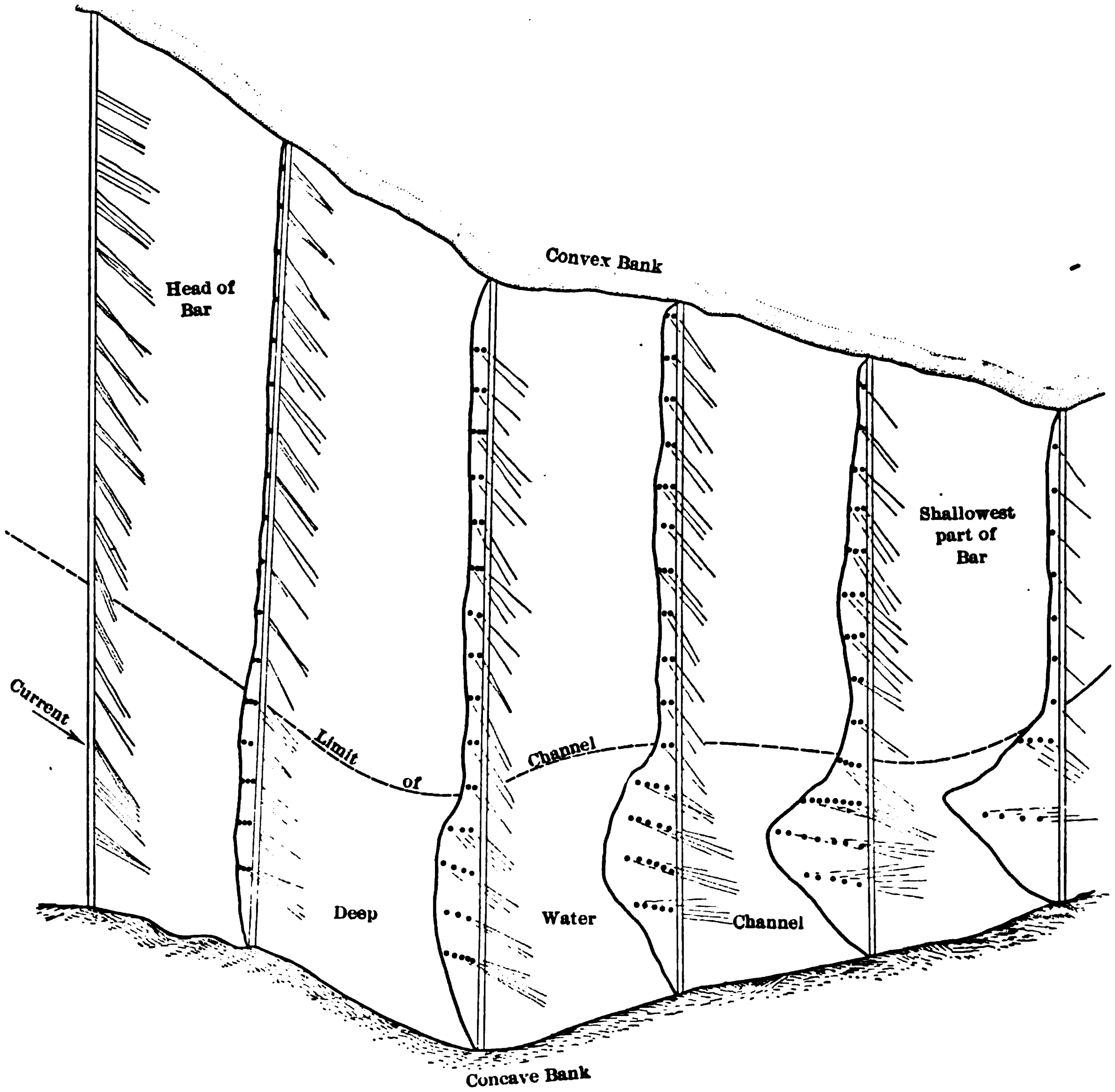



FIG. 3a.—Illustrating the Movements of Threads of Flowing Water. From a photograph of a model by M. N. de Lelavski, based on observations by floats on the Dnieper River in 1902. The movements were represented by pins placed in pieces of glass which were shaped to conform to the water surface and to the contours of the bed.

When the falling river begins to remove the deposits from the shoals it has to deal with material of varying compactness. If the flood has been a prolonged one, the matter first deposited will have become well settled and the water may have trouble in loosening it. It will then carry off all the lighter portions, and in so doing the local currents may obtain a new direction, other layers of soft or of hard material will be uncovered,

and the process of change of flow and contour will proceed without interruption until the water has become too low to effect further erosion. On streams of pronounced alluvial character, such as the Missouri or the Arkansas rivers, the navigation channel during such periods may shift at certain bars from one bank of the river to the other in a single day. The change is frequently accelerated by the stranding of snags or drift, when there results an immediate shoaling below them.

The scouring is a slow process, and if the river falls rapidly the bar may become an obstruction to boats before the water can open a channel through it. On that portion of the Mississippi where navigation in low water is maintained by dredging, a rapid fall is a sure indication of approaching trouble, while with a slow fall very little dredging may be required. Thus in 1903 soundings on a bar about 40 miles below Memphis showed a depth of only 11 feet at a time when the stage was 16 feet above low water, so that if the river could have fallen suddenly to its low stage, there would have been a bar 5 feet high extending from one bank to the other. The bar was of course scoured down as the water receded.

The scour produced by the slope over the shoal necessarily decreases after the water has receded beyond a certain point, not only because the volume and swiftness of the flow decrease, but also because in the erosion the particles are carried away from the crest of the shoal and are deposited at its lower end, thereby flattening the slope and causing a further decrease in velocity.

The effect of scour is supposed to vary as the square of the velocity, but it is also very largely dependent on the depth of the flow. Thus a river with a discharge 10 feet deep and a velocity of 4 feet per second, pressing on its bed with a weight of 625 pounds per square foot as it passes along, tends to produce a good deal more scour than a brook of equal velocity but only 1 foot deep, which presses only with a weight of $62\frac{1}{2}$ pounds per square foot.

*Eddies and Currents.*—The flow of a river during these periods of low water is tranquil; the current in the pools moves slowly and steadily, and it is only at the shoals that disturbances of any moment exist. In time of flood, however, the conditions are reversed; the main channel through the bends and often far down the crossings is alive from depths to surface with swirls and currents, the most remarkable of which, partly due, perhaps, to the reactions from the sand-waves described further on, are the "boils" or vertical eddies, an upthrow of water to the surface as though great springs had broken from the depths. We have been informed that on the Mississippi in high floods these boils at times spread half way across the river, and may reach a height at their center of some 5 feet above the normal surface of the flow. The resulting disturbance of the water is tremendous, and very dangerous to boats, especially when towing, and more than one case is known where craft have been caught and sunk by the sudden cross currents.

In addition to these vertical eddies two other kinds, rotating horizontally, are

met with. These are known to the Mississippi engineers as "suction" and "pressure" eddies, and are shown in Figs. 4 and 5, the former kind being the more common. They are due to the presence in a caving bank of a hard stratum which is eroded slowly and which gradually forms a "false point" projecting into and deflecting the current. Where the point is of considerable size a violent eddy is formed under its lee, with a surface velocity usually much greater than that of the main stream, and a vortex at the center capable of engulfing good-sized drift. These suction eddies occur at times on the Mississippi with lengths of 800 feet and 200 feet across. They have been measured on the Ganges having a diameter of 200 feet, with velocities of 11 feet per second at the circumference, and with the water at the center 3 feet lower than at

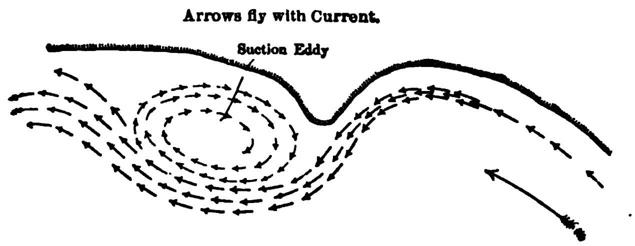



FIG. 4.

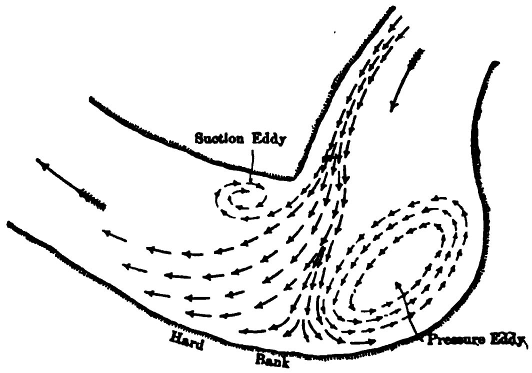



FIG. 5.

the outside. The deepest point under these eddies is not usually at the center, where the water moves less swiftly than at the circumference, nor is it near the bank, where the material is constantly falling in. It will usually be found along the river face or towards the downstream side, and may run from a few feet more than, to two or three times the general depth of the river, depending on the location and the softness of the bed.

If the river is running swiftly, the center vortex is at times swept bodily downstream by the current, and the conflict of the inflowing water and the swirling of the eddy produces the appearance of one of the "boils" just described, as though the water was flowing up from below. A new vortex then forms and the phenomenon is repeated until conditions change. This type of eddy has an important influence on

the caving of the banks, since the point may be of soft material on its downstream side, though hard above. In this case the erosion works slowly upstream under its shelter, and may seriously threaten any protective works in its immediate vicinity.

The pressure eddy is caused by an abrupt change of direction and occurs under conditions similar to those shown in Fig. 5. It is usually accompanied by a suction eddy on the opposite bank. It rotates with comparative slowness, and moves with less velocity than the river, and hence it may form large deposits. In the reach close to Memphis, Tennessee, a pressure eddy has been observed 2000 feet wide and nearly a mile long. It ebbed and flowed in regular periods of about thirty minutes each, the flow being reversed during each period, and a gauge near the lower end of the eddy showed that the water rose and fell about 6 inches with each pulsation.

To such restlessness of the water is largely due its power to carry sediment; particles beginning to settle are caught up again and hurried onward in the confusion of the flow, and it is not until a portion of the river is met with where the flow becomes more tranquil, such as a wide and comparatively straight reach or the shallow water along a convex bank, that the sediment can find a resting place. Samples of water taken from a bend and others from a quiet reach will show a noticeable difference in the amount of matter in suspension, and in muddy rises the variation in the clearness of the water at the surface is plainly visible. The slackening of the current over the crossings is the prime cause of the shallow depths there; the lessening of the swirls and eddies is the secondary one. The more the velocity is increased the more disturbed will the water become, and the greater will be its power to transport the matter eroded from its banks and bed, and per contra, as the velocity decreases, so do the swirls and the power of transportation. The relation between the velocity of a river and the amount of its sediment, however, appears to be variable, since the irregularities of flow just described have a great influence upon it.

*Sand Waves.*—In swift water the bed appears to be corrugated with sand ridges or sand waves, similar to, and produced by similar reactions as, the sand ripples to be seen in shallow currents. They show the same long upstream slope and short downstream one, and the eddies of the flow swirling over the former catch up material from its surface and drop much of it behind the crest, eroding the one side and building out the other, so that the wave travels slowly downstream with its flanks trailing a little to the rear. On the Mississippi such waves have been measured in deep swift water with a height of 20 feet, and moving at a rate of 80 feet a day. On reaching shallower water the movement lessens, accompanied by a tendency to build up the head of the succeeding shoal, though this effect, compared with that due to the deposit of suspended sediment, must be slight. Below the shoal the movement again begins and augments as swifter and deeper water is reached, increasing the tendency in the bends to scour in floods. A change of stage or of velocity seems to cause a corresponding change in the height and rate of travel, and the deeper and stronger the current the larger

and farther apart are found to be the waves, just as waves in the open sea increase in size and distance with the power of the wind.

Some investigations in connection with the formation of sand waves were made in 1883 in connection with certain works proposed in the Mersey for the Manchester Ship Canal.* The experiments were made in a trough with glass sides, using the light sand found in the estuary. It was found that the particles began to take the form of sand waves under a surface velocity of 1.3 feet per second, and at 1.5 feet these became well defined, and moved forward at a rate of $\frac{1}{2160}$ of the surface flow. At a speed of 2 feet the rate became $\frac{1}{480}$; at 2.1 feet the particles began to be lifted instead of being rolled along; and at 2.9 feet the water and sand became indistinguishably mixed. It may be inferred, however, from the presence of the sand waves observed in the Mississippi during floods, that the volume of the water has as much bearing on their formation as the velocity of the flow.

*Erosion.*—Referring again to Figs. 2 and 3, p. 14, it will be noticed that the tendency of the water is constantly to work into the bends, eroding the concave bank or "bend side," and building out the convex bank or "point." This is due partly to the centrifugal action of the flow and partly to the effects of the moving sediment. The former flings the main current, and therefore the main velocity, against the bank, so that while the river may be flowing at 5 feet per second close to the concave shore, at the same distance from the other shore its velocity may be only 1 foot per second. Hence the water traveling over the point will drop much of its burden there, and this deposit will gradually crowd the river further into the bend. This same water, at the commencement of the next bend, finds itself on the concave side with only part of its possible load, and therefore attacks the bank in turn and gathers a new burden to be deposited further down.

Measurements show, however, that the width of each bend remains about constant; as fast as one side is eroded the other is built out, the river holding to the area needed to carry along its sediment. It is deduced from this that the greater portion of the sediment is washed in from the upper portions of the watershed of the main river and its tributaries. An illustration is given in Fig. 5*a*, showing the gradual shifting of the Mississippi at Memphis caused by the erosion of the point just above the city. In fifteen years the left bank filled out until its area had been increased by 106 acres, with a maximum increase in width of 2300 feet, while parts of the former channel silted up 45 feet in one year. This alteration of the flow necessitated the protection of a long distance of the right bank, as shown in the figure.

The erosion, if the banks are steep, is assisted by dry weather and by frost, during which the exposed earth crumbles and falls, sometimes in particles, sometimes in masses. When the next flood comes it finds a large amount of light pulverized material which it quickly removes, and then begins again its attack upon the bank.

* Proceedings, Inst. C.E., vol. cxviii.

The tendency to shift the bed, due to this erosion, is always at work, and it is interesting to compare surveys of a river made ten or twenty years apart, and to note the degree of change. Figs. 6 and 7 show such an example in two characteristic stretches of the Rio Grande, which forms for some 800 miles the boundary between the United States and Mexico. This stream is notorious for its unstable bed, so much so that an agreement was made between the two Governments that when the river cut off any portion from one country, thus transferring it in fact to the other side of the river, its nationality should remain unchanged.

It may be noted that although the river tends gradually to move its bed downstream by the erosion of its banks, it is claimed that the shoals and deep water

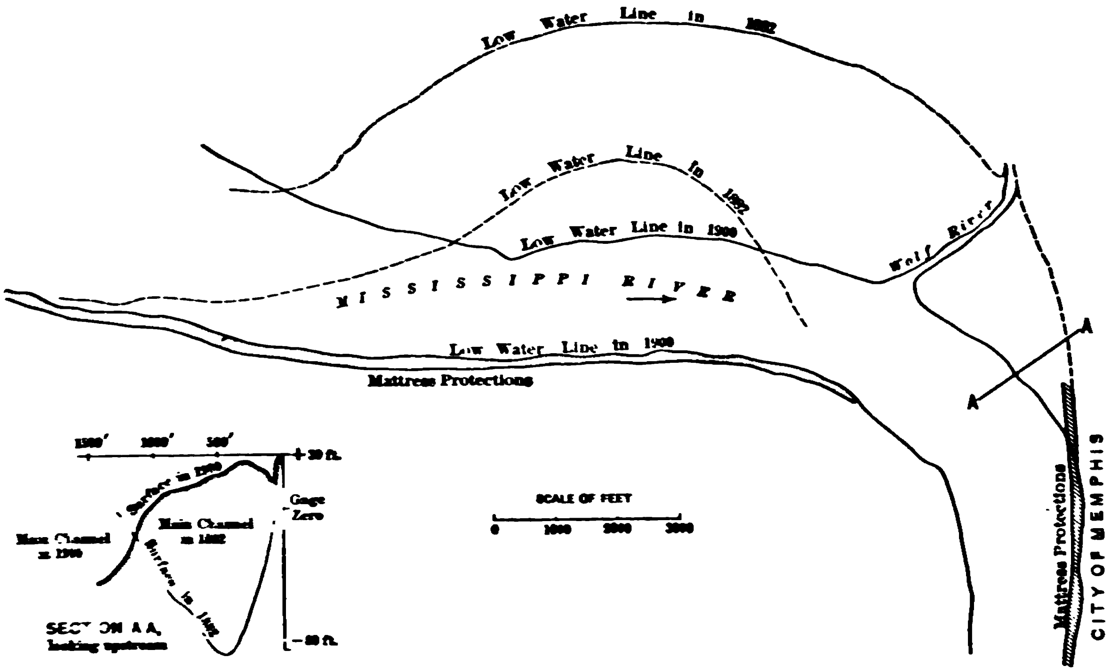



FIG. 5a.—Changes in the Mississippi River at Memphis, Tenn.

gradually move upstream.* Such actual data as are available, however, do not appear to confirm this theory. Thus in the rivers Waal and Merwede in Holland it was found that the shoals and the intervening channels moved downstream, keeping always the same relative positions. The rate of change varied from 300 to 500 meters per annum, and one shoal would reach the place formerly occupied by the next shoal in a period varying from six to twelve years.†

A more noticeable fact is that when a river shifts its bed through gradual erosion, the islands are usually carried bodily downstream, the lower ends building up as the upper ends are carried away. Thus on the Paraná, the island of Espinillo, opposite

* Mémoires, Institut de France, vol. xx, M. Dausse, and Journal of the Geological Society, London, vol. xliv, R. D. Oldham.

† Transactions Am. Soc. C.E., vol. liv, p. 412.

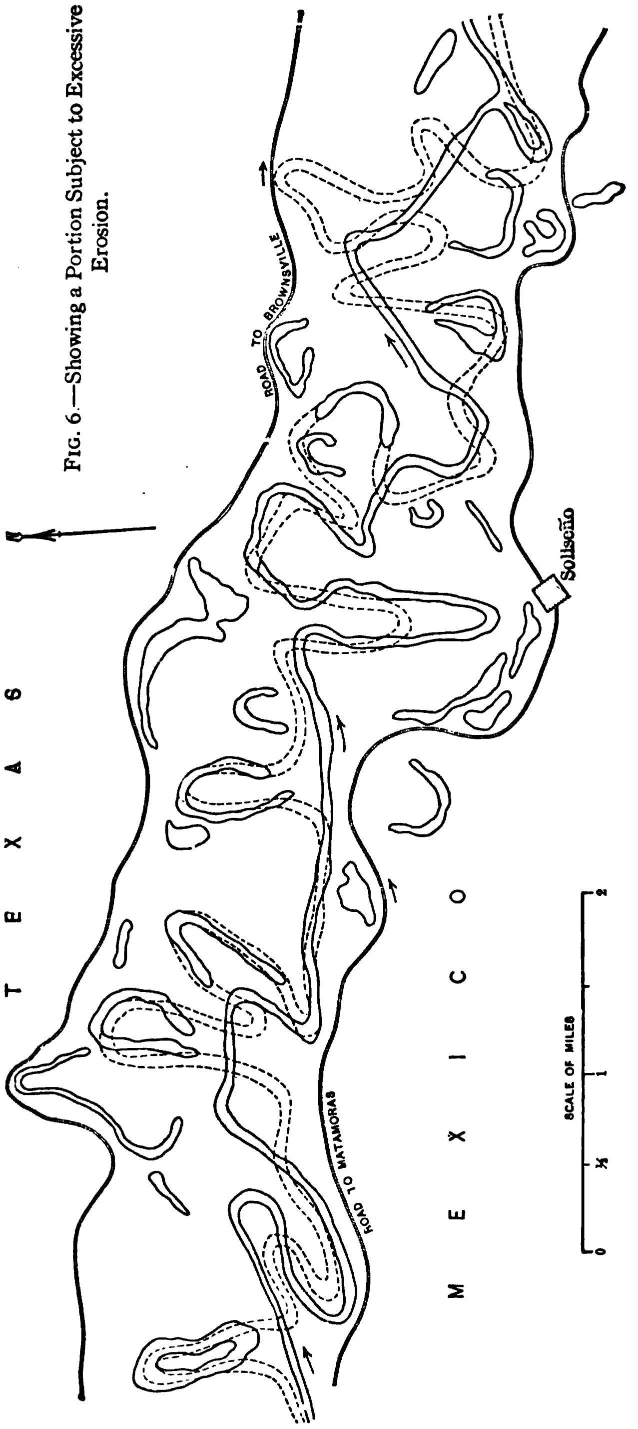



FIG. 6.—Showing a Portion Subject to Excessive Erosion.

CHANGES OF THE RIO GRANDE DEL NORTE. (From Surveys of the United States and Mexican Boundary Commissions.)

Bed of River in 1853: ---------
" " " 1898: - - - - -

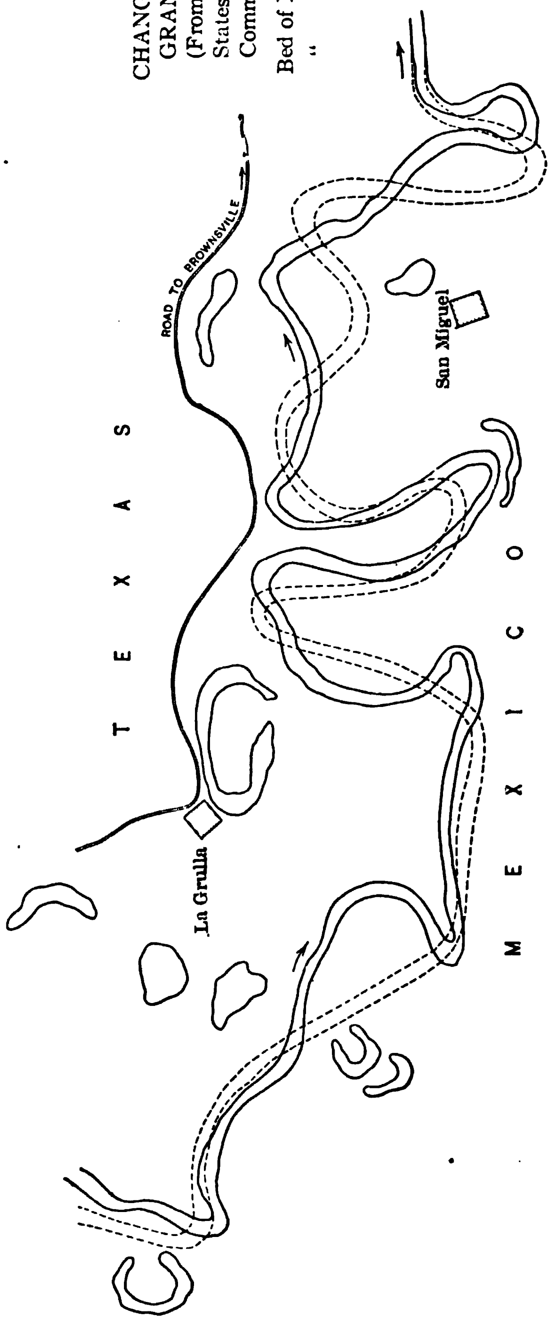



FIG. 7.—Showing a Portion Subject to Moderate Erosion.

Rosario, moved downstream $2\frac{1}{2}$ miles in fifty years, shifting its axis at the same time, and a comparison made by Mr. E. L. Corthell of maps of various dates from 1525 to 1885 showed the constant though gradual evolution of present conditions. The early maps showed an estuary extending almost to Santa Fé, but as time went by the islands began to appear and new shoals to form, until the stream had built up the well-defined course it now holds. The history of the river is supposed to be similar to that of the lower Mississippi, whose estuary, once extending far inland, became gradually filled up with detritus.

It has been estimated that the natural forces causing the removal of material through erosion by rains, etc., reduce the surface of the watershed of the Po 1 foot in 700 years; of the Rhone and of the Hoangho, 1 foot in 1500 years; of the Ganges, 1 foot in 2400 years; of the Mississippi, 1 foot in 5400 years; and of the Danube, 1 foot in 6800 years.*

*Caving.*—Erosion is the primary cause of these ceaseless changes, and its amount is greatly increased by the slipping-in or caving of the banks. This may be due to the water eating into their base; to the presence of a layer of sand which is quick'y carried out and leaves the soil above without support; or to a sudden fall of the water which leaves the saturated earth with too great a load.† The bank will then fall in large masses, and lie shattered and exposed to the current. Although this caving protects temporarily the parts still standing, the broken condition of the fallen soil creates fresh eddies, and the erosion usually proceeds rapidly until the bank itself is again exposed to the attack.

Caving usually goes on at all stages except that of low water, when it is rare, but it is of course at its height during floods. It is usually worse on a falling river, as the banks are left saturated, but its maximum at different points may occur at different stages. In sharp bends the erosion may continue all the time, and it sometimes occurs at shoals owing to a bar causing the water to impinge against the bank, and it has also been found to result from dredging. Eddies below sharp points are a frequent cause of caving and erosion, as their action is often more violent than that of the main river. (See p. 21.) At Fletcher's Bend on the Mississippi, about 160 miles below Cairo, a single flood in 1898 cut out a pocket 800 feet long and 250 feet wide, with depths of 40 feet where the year before had been dry land, and at Hopefield Bend just above Memphis the bank was cut back by direct attack 750 feet in one year.

Where a levee system exists the falling of the banks often makes it needful to rebuild the levees further back. Thus the Mississippi Levee Board was forced to incur in one season an expense of about a million dollars in order to construct loops to replace levees which were threatened with destruction by the caving of the banks.

* Sir A. Geikie, Geol. Magazine, vol. v.

† The processes and effects of erosion, caving, cut-offs, etc., are described in further detail under Chapter V, "Protection of Banks," See also p. 44.

It is interesting to note in this connection the supposed influence of a cosmic law, adduced by several authorities, that there is a tendency in all streams to erode the west bank slightly more than the east one, owing to the rotation of the earth. Gilbert states that on the Mississippi the difference is about 9 per cent, and it is claimed that certain of the measurements of the Mississippi River Commission tend to confirm this observation.* On the other hand we have been informed that on the shifting rivers of northern India, flowing principally through beds of sand, no record of any such tendency has been traced by their engineers, and that on the Indus there is to be found a former river-bed far to the west of the present one. The theory, while plausible, appears to lack final proof.

*Cut-offs.*—A river appears therefore to tend to shift gradually its entire bed, and when a bend has become almost a complete curve, the river breaks through the intervening neck of land and forms a cut-off. (Fig. 2 p. 14; figs 6, 7, and 7*a*.) This may occur either through a flood overtopping the bank, or simply through the erosive action at each side during moderate stages. Usually the former is the cause, since the distance across the neck is short, while around the bend may be many miles, and the steep path open to the water as soon as it overtops the bank results in a swift erosion and a rapidly increasing depth in the new channel. During the flood of 1903 the distance around the bend at Leland's Neck near Greenville on the lower Mississippi was 13 miles, while across the neck was only 3000 feet. The difference in the elevations of the water surfaces at the upper and lower sides was 4.7 feet, giving a rate of fall when the water overtopped the bank of some 9 feet per mile. Owing, however, to the existence of a spur levee and to the receding of the flood, a cut-off was temporarily averted. When a cut-off commences under such conditions, the flow naturally endeavors to follow the shorter route, and as soon as a good volume can find a passage, an appreciable lowering of the upstream water-levels and a raising of the downstream ones takes place. This is due to the steep slope through the cut-off being able to pass a given flow more rapidly than the old channel or the channels above and below, and as a result of the changed levels the current through the old channel slackens and begins to deposit its sediment. The greater portion is let fall at the first checking of the flow, that is, in the old channel just below the upper end of the cut-off, gradually creating a barrier which tends to increase the flow through the new outlet. (The phenomena are similar to those observed when a crevasse opens through a levee; the same lowering of the neighboring water-levels takes place, and a similar tendency of the river has been found to deposit sediment temporarily in the neighboring channel, due to the slackening of the flow caused by the break.) Meantime, while the stream is busily at work blocking up the entrance to its former channel, the flow of water through the cut-off is enlarging the new path and eroding a vast quantity of earth which it must get rid of as soon as its speed is checked. Much of the burden is let fall immediately below; some

* Transactions, Am. Soc. C.E., vol. li, p. 401.

is deposited in the main channel, where, by raising the bed, it tends to reduce the slope and to restore a temporary equilibrium, and some is carried by the eddies into the slowly dying current of the old channel, and builds up across the exit a bar such as is growing at the other end. When the flood subsides there will usually be found a


FIG. 7*a*.

bank of deposit completely blocking each entrance, and sloping gradually back from the river. These entrances are built up by the river year by year, a growth of shrubs and trees springs up, and in the course of time no trace of the old conditions is visible along the now continuous bank. The lagoon itself, however, the former channel, may remain for generations, or until the stream breaks into it at some new point. We have

met with cases where the river had built up the old entrances with banks more than 40 feet high (which it very rarely overflowed), and to the general level of the bottom land, so that from the river not the slightest trace of former conditions could be seen. About fifty or a hundred yards from the bank, however, a slight depression would commence, and following it through the forest growth down a long steady slope, the remains of the old channel would be found, filled with water, and still retaining the width and curvature of the main river bed.

*Local Strata of Hard Clay.*—These cut-offs are frequently the origin of the layers of hard clay which unexpectedly occur in alluvial banks, and which often have an important local bearing on the configuration of the bed. Their growth has been described as follows:*

" It often happens that, for a distance that may be only 100 yards or may be a couple of miles, a river finds itself flowing past a comparatively unerodible clay bank; not a piece of non-alluvial bank of older than river formation, but a clay bank formed *bona fide* by the river itself. The way in which such local clay lumps may be formed will be understood when it is considered that, on the occurrence of an extensive cut-off, the old bend which is left will, ordinarily, silt up very quickly. But the old bend does not always silt up at once: it may take many years to do so, for perhaps it gets closed immediately by a bar at the upstream end, and for the rest of its course it forms for many years a deep still backwater. If this backwater is separated from the lately formed cut-off channel by a high piece of rapidly silting land, covered perhaps with long grass and low jungle interspersed with cultivated patches, very little silt-laden water passes in and out of the backwater. What water does get into it, and that perhaps for only a few days in the year, either has dropped its heavier or sandy particles already, near the downstream entrance to the backwater, or else is only the top layer of the flood-water, a foot or two deep. carrying very little solid matter and that only of the finest clay. Thus very little but the finest clay gets deposited in such a backwater; and if only it can preserve its integrity long enough—for instance, if through various strokes of luck it should fail again to become main river for a period of fifty to eighty years—it in due course becomes silted up, but only with fine cohesive, putty-like potter's clay of great density, perhaps indurated with a little carbonate of lime. Sooner or later the time comes when the intermediate strip of land gets eroded, and the river finds itself impinging on the patch of indurated clay, which meanwhile has filled up the entire shape of the original lagoon of many years ago as wax fills a seal. The patch of clay so deposited may be 100 feet deep below low water stage if the original bend was so deep; it may be 300 feet wide if the original bend was so wide.

" Now it stands to reason that a river flowing past, or even impinging at an angle against, such masses of clay as this, at velocities good enough to erode less cohesive material; will make comparatively little impression on them. Lumps and even large

* "River Training and Control on the Guide Bank System," Francis J. E. Spring.

areas of such material are known to have stood for twenty or thirty years, in the face of dangerous looking impingement, and yet to have been scarcely at all eroded. But if it should have so happened that the original lagoon which formed its mound was not very deep at the moment the 'cut-off' established itself, the clay lump itself will be found to be correspondingly shallow and to rest on the ordinary non-coherent river sand below. Now if the river should see fit to scour deeply along this clay, it may easily underscour it from below, and great lumps of it will cave in. The character of this caving is different from that which goes on in high banks composed of less coherent material, and the clay débris, instead of being carried away in suspension, serves to protect the exposed underlying sand strata, to some extent, as it rolls down. Particles of this sort of clay, in all sizes, from that of a grain of shot to that of an ox, are often met with in boring through sand, and are liable to be mistaken for continuous clay strata—a very serious mistake to make. The author has often dredged them out with grabs, from depths down to 50 feet below the river-bed. When got out they are generally found to be almost globular, from having been rolled along the bottom perhaps for many years and many score of miles; inside they are like perished black rubber and cut like cheese; they are usually devoid of even a grain of sand; and they do not disintegrate when allowed to stand for even a long time in water. Many clays, however, found on river banks, even when apparently as hard as the special lagoon-deposited clays, will disintegrate into mud if left standing a few hours in water. It is the former clays and not the latter which every here and there, on some rivers, hold the current at one point for years together. But however coherent, or however deep and thick they may be, they must be eroded sooner or later under continuous river impact."

Mr. T. H. Holland, Director of the Geological Survey of India, has made chemical analyses of these clays and of the silt carried in suspension in the neighboring river, and found, as would be expected, that their composition is practically identical.

*Slope of Banks.*—Another feature of alluvial rivers is that the adjacent lands slope away from, and not towards, the main stream. This is due to the same causes that create shoals in the crossings; the silt-laden water, which moves rapidly when flowing in the channel, rises over the banks during floods, and as it passes away from the main current loses its velocity and deposits a proportionate part of its burden. The heavier portions are of course deposited at the first checking of the flow, which occurs as the bank line is passed, and hence the bottom land close to the stream gradually builds up and creates the slope referred to. In valleys with narrow bottom lands the result is a slough or drain close to the hills which returns the water to the main channel further down, one slough succeeding another along the valley; with those of wide bottoms, the result is a secondary basin drained by other rivers, such as the Yazoo and St. Francis Basins of the Mississippi. The river thus actually flows between embankments or levees of its own building (Fig. 8), which it overtops in time of flood, but which it tends continually to raise, and which would eventually restrain high water but for

their being continually subjected to erosion. In extreme cases its bed may be found at a considerable elevation above the surrounding country, as with the Reno, one of the most dangerous of the Apennine rivers, which flows in places at an elevation of more than 30 feet above the adjoining country. On the lower Mississippi the land in some places slopes away from the river more than 10 feet in the first mile. Similar conditions are often to be seen where the deltas of rivers have built far into the sea, and in course of time they may lead to the stream's forsaking its old channel and seeking the lower levels by a shorter route.

It may be noted also that the alluvial features of a river are usually most dominant along its middle and lower portions, since the upper portion generally flows through land of more stable composition, or through valleys too narrow to permit much change of bed. As the river approaches the sea, its slope becomes less and its channel deeper, the latter being a result necessary to carry its volume of flow, and its effects on its banks are less destructive, so that in what is actually the most alluvial portion of the stream, the sharp bends and cut-offs met with above, and the great range of floods and sudden variations of depth, become rare. If, however, the river falls into another alluvial stream, as is the case with the Missouri, its features of instability usually

FIG. 8.

continue up to the confluence, since the slope and flood range of its basin still remain considerable.

*Grinding of Sediment.*—In order to illustrate this portion of the labors of a river, a stream of ordinary type may be taken and the effects of a general rainfall traced. In the upper portion the valley will probably be narrow and the slope steep, and the eroded matter will consist largely of coarse sand and gravel washed in from the hills. Every little tributary will bring in its share, much will be washed in from the banks by the rain, and some eroded by the river. The steep slope will permit of a high degree of saturation for the general flow, which will carry in suspension the sand and silt, while the gravel will be rolled along the bottom. If the gravel is too heavy, or where rough stones are washed in by a tributary, the current cannot shift them, and the pieces form a bar which grows until it may force the river to the far side of its valley. Part of the matter carried on will be left on the shoals or on the points, the lighter portions continuing their travel until parts of them also find a resting place. In the journey, however, the coarser particles are being constantly ground finer, and require less and less effort to transport them, and many will thus be carried entirely out of the river. As we continue down the stream new matter is constantly being brought in, but as the valley widens less gravel reaches the main bed, since the slope of the tributaries as they approach the river becomes correspondingly less. The process of grinding still goes on, the sediment

becoming finer and finer but always tending to keep equilibrium with the carrying force of the water, until near the mouth (unless the slope of the river remains comparatively steep) little else than fine silt will be found in the deposits or in suspension in the stream.

This process of grinding has a very important effect upon the river by reducing the coarseness of the detritus. If the latter remained unchanged, the slope of the stream would be as great near the mouth as along the upper portions, but as the detritus is ground finer, so the slope decreases. (See Fig. 1, p. 8.) As to the rapidity with which the grinding progresses, no specific data appear to exist, but results show that it must be considerable. Thus the bed of the Ohio River near Pittsburgh consists largely of coarse sand and gravel; 300 miles down the river the proportion of sand and small gravel becomes much larger, while near the mouth the gravel disappears almost entirely. Similarly the upper portion of the Tennessee River has chiefly coarse gravel; the lower portion has only fine gravel or sand. The maritime portion of the Rhone flows through a bed of fine sand and silt, while further up the bed is of gravel, much of it very coarse. Lord Avebury quotes an experiment by Daubrée in which pieces of rough granite and quartz were placed in a horizontal cylinder partly filled with water, and the vessel was then revolved with a peripheral velocity of one meter per second. After a movement equal to a travel of 16 miles, all the angles of the stones were perfectly rounded, and the pieces had the shape of ordinary pebbles.

**Effects of Ice.**—Ice sometimes produces important local changes of bed by "gorging" or piling up and so temporarily obstructing the flow and causing it to scour a passage under or around the obstacle. These gorges are frequently due to low islands or shoals dividing the natural axis of flow; the floating ice lodges on them and the floes begin to collect until a mass has piled up which may completely dam the river. The gathering water finally bursts the mass, when the pieces are carried downstream in the rush and may form another gorge further down. In thickly settled localities property is often flooded and much damage caused by the abnormal raising of the water levels, as in the Mohawk Valley in the State of New York, where the main-line traffic of the New York Central Railroad has occasionally been blocked for several days by the water and masses of ice being forced over the tracks.

Where the channel is regular and of good depth ice-gorges very rarely form, but where it is shallow or broken by islands the evil will aggravate itself by the constant forced changes of the flow and of the shoals. On a certain portion of the Allegheny River which had always been subject to ice-gorges and was full of shoals and islands in consequence, a comparison of old maps showed that in the course of years the islands had shifted and moved downstream many hundred feet. Another cause of ice-gorges is a pool of still water which becomes frozen over and remains as "field-ice." A moderate freshet will loosen and carry out the ice from the steeper sections above and pile it under and on top of the unbroken field-ice of the pool until the river is choked.

Some of the numerous outlet channels of the Rhine were formerly a cause of much trouble from ice, owing to their shallowness, but vigorous dredging aided by levees was applied to deepen and improve the channels with the result that the ice now obtains an unimpeded exit to the sea.

**Low- and High-water Levels.**—The low- and high-water levels of a river are both subject to fluctuation, and the construction of works of improvement may change the conditions to such an extent as to affect seriously levels previously established. As a rule the raising of the high-water level will not seriously affect navigation, but it may cause injury to the adjacent lands. As a country is cleared and cultivated and the forests removed, the water carries great quantities of material into the stream, because much of the rainfall will reach the river rapidly under the changed conditions, and the result may be that floods become more frequent and violent, and the banks of the river and its tributaries will be cut away and carried into the stream. This action may be aggravated by the employment of splash-dams and the removal of obstructions in the smaller streams for the purpose of carrying to market the neighboring timber. This constant inflow of material has the result of gradually filling up the pools and reducing the depths for navigation. In one instance which came under our observation, surveys made in 1875 of a river of steep slope, a tributary of the Ohio, showed the usual series of deep pools separated by shoals. After that time the country was stripped of much of its timber, land was cleared, splash-dams for bringing out logs were built and operated in many of the side streams, the river itself was improved by clearing its bed and confining its channels, and by 1895, a deep pool could not be found. They had practically all filled up, so that where, twenty years before, the water was 20 feet deep at the low stage, it became less than that number of inches. The pools had not only filled up, but the low-water level and river-bed had been raised in the lower portion of the stream, the tendency being to form a bed with a regular slope, and in many cases these grade-lines had obliterated the smaller shoals entirely. This new bed was in many places entirely above the old low-water level. Thus in the lower part of the stream a gauge was set with its zero 9 inches below low-water mark. Ten years later the new low-water mark was 2 feet above the zero, with a less actual discharge than at the time the original gauge had been set, and in the same period all the shoals and pools within 10 or 15 miles of the gauge had disappeared, and regular slopes had been established This action was not due to the construction of any improvement works in the vicinity but solely to the excessive inflow of alluvial matter. Similar but much more extreme conditions were brought about on some of the rivers of California, due to the immense amount of gravel and sand washed into them by hydraulic mining operations. Parts of the beds filled so that navigation became impossible, and much land was ruined by the permanent raising of the flood levels.

In the study of a project for open navigation allowance should be made for possible variation between the approximate low water which is known and the extreme low

water which may occur under rare conditions, but the position of which for that reason may not be known. The approximate stage is generally called ordinary or mean low water, and represents the depth ordinarily reached by the river at periods when there is just sufficient water for navigation, and when boats must therefore proceed with more care and less speed.

The highest navigable stage is also one of importance, particularly in fixing the level of bridge floors. It is seldom practicable or indeed necessary to raise the masonry of works of navigation above the level of the highest navigable water in America, although this practice obtains in parts of Europe where the flood variation is not so great. The level at which navigation ceases necessarily varies, and can be increased under certain conditions if boats are provided with more powerful means of propulsion. When a river reaches a very high stage, however, navigation may become difficult, and even dangerous, on account of the increased velocity of the current and of the uncertainty of the directions of its forces. Ordinarily navigation ceases when the water overflows the banks, but this may vary with circumstances and at different points on a stream according as the banks are high or low, or the river comparatively straight or crooked.

It is also of importance to know the height of the greatest flood. Like conventional low water this level is subject to change. American rivers are in general subject to floods of much greater height than are those of Europe, where floods on large streams rarely exceed 20 to 30 feet above ordinary low water. The Ohio River has had a number of floods exceeding 50 feet at Cincinnati, and that of 1884 rose a little over 70 feet. Where rivers are fed by lakes they are not subject to such great extremes of level. This is particularly noticeable in the St. Lawrence River, whose level varies but slightly. To a certain extent the same is true of the Rhone, the Rhine, and a number of other streams.

**Velocity of Flood Crests.**—The velocity with which the crest of a flood-wave travels varies with the slope and other elements of the river. On the Loire the average rate is from 2½ to 4 miles per hour, and the local rate over a stretch of about 55 miles below the Bec d'Allier, in which there are no affluents, is 2.9 miles per hour, being the same for low and for high floods.* On the Marne the rate is from 1.6 to 2.1 miles per hour, while on the Mississippi the crest of the flood which overflowed the banks in 1901 traveled the 230 miles between Cairo and Memphis at a rate of 1.9 miles per hour, and the 370 miles between Memphis and Vicksburg at 1.7 miles per hour. This rate was considerably less than the general velocity of flow, the retardation being presumably due to the effects of overflow. With floods which do not overtop the banks the rate of travel is much faster, and a mean of several observations indicated a crest velocity of about 3.8 miles per hour between Cairo and Memphis and of about 3.4 miles per hour between Memphis and Vicksburg. (These portions of the river are shown on Pl. *1a*.) More extended observations have shown that the rate of

* "Rivières à courant libre," p. 65

passage varies considerably in the different portions of the river. Thus with a moderate or "bank-full" flood the rates between Cairo and Baton Rouge (833 miles) were found to vary from 2.5 to 5.6 miles per hour, with an average of about 3½ miles per hour, while on the 21 miles just below Cairo the rate was 8.9 miles per hour. It was found also, taking the entire distance between the two cities just named, that the average velocity of the flood crest corresponded very closely to the average mean velocity of the flow, while below Baton Rouge the crest appeared to be accelerated and to assume the motion of a wave of transmission, its speed at Carrollton (about 120 miles from the sea) reaching 22½ miles per hour. By comparing isolated stretches above Baton Rouge, however, it appeared that the crest traveled faster than the mean flow velocity in some cases, and more slowly in others.*

**Rate of Rise.**—This varies with the size of the river, the slope of its basin, and other conditions. A sudden and heavy rainfall on some of the tributaries of the Ohio has been known to cause a short flood-rise in these tributaries at the rate of 3 or 4 feet per hour, and occasionally at even a greater rate, as on the Kanawha River, where a thunderstorm in September, 1908, caused a local rise of 4 feet in twenty minutes. The upper Ohio itself has occasionally risen 2 feet per hour for an hour or more. Records at Pittsburgh show a rise in 1903 of 15.8 feet in sixteen hours, and others of 8.7 in four hours, and 19.8 feet in twenty-four hours. Ice-gorges breaking above the city have caused rises at a rate as high as 5 feet per hour. At Cincinnati the fastest rise recorded for the Ohio was 14 feet in twelve hours, equivalent to 14 inches per hour. Similar records for the upper Mississippi at St. Paul give 3.8 feet in twenty-four hours, or 2 inches per hour, and at Rock Island 6.1 feet and 3 inches respectively. At St. Louis the fastest rate on record is 0.41 ft. per hour; at Vicksburg, 0.33 ft., and at New Orleans, 0.08 ft. On the Brazos River in Texas one flood rose 25 feet in a single night and 44 feet in fifty-eight hours.

**Characteristics of the Mississippi.**—The application in Nature of the principles which have been described in the foregoing paragraphs may be illustrated by the following description of the Mississippi:†

"The Mississippi is above all things a silt-bearing stream, flowing through a bed of its own creation. It is never clear, even at its lowest; and at its floods it is charged with sediment to an enormous extent. This involves the following consequences: the banks are low and subject to overflow by the floods greater than the mean; the banks and bottom are friable, and, therefore, liable to degradation and erosion. The channel is shifting and unstable, and subject to obstruction by shoals.

"The Mississippi, like all other streams, is composed of tortuous bends, which

* Annual Report, Chief of Engineers, U. S. Army, for 1892; pp. 2911 and 2912.

† Transactions, Am. Soc. C.E., vol. xx, Wm. Starling. A map of a portion of this river will be found at the end of the book as Pl. 1a.

modify, in the most striking manner, the laws of its flow. In the portion of its course which we are considering, it runs through a bed of comparatively recent origin. It is seldom that it encounters a portion of any formation older than that which constitutes its present flood-plain. At a few points the banks consist of bluffs of a quaternary formation, called by geologists loess, from a similar formation on the Rhine; at a few more, its bed trenches on another quaternary formation called the "Port Hudson" clay, not very dissimilar to that of the present alluvium, and deposited by the river itself in its ancient state, at an epoch anterior to that of the present stream. It has been a much mooted question how great a part of the present bed is occupied by this older and harder clay; but the investigations of the Mississippi River Commission appear to point to the conclusion that the proportion is small.

"The alluvium composing the banks and the bed is extremely variable in composition, being sometimes nearly pure sand, sometimes nearly pure clay, sometimes made up of both ingredients in all proportions. These different qualities of soil are disposed in strata of every variety of thickness. Thus the banks are friable, but in very different degrees. Usually, however, even the toughest clay is underlaid by strata of lighter material, which, as will soon be seen, diminishes the resisting power of the stronger earth.

"The theory of the completed flood-plain of a river is well known. The sediment brought from the torrential and fluvial parts is deposited on the bottom, and, by overflow, on the banks of the stream in its alluvial part; thus raising the upper portion of the flood-plain, and steepening its slope until the velocity of the water becomes sufficient to carry the load without diminution to the sea. Not every flood-plain is completed. Some are yet in a state of transition and have not attained stability.

"Bends introduce a disturbing element into this process. The current, flowing with the normal, or even with a slightly accelerated, velocity, encounters the resistance of the bank, placed at a somewhat abrupt angle to the course of the stream. A portion of its energy is directed against the soft material composing the bank, promptly dislodges it, and carries it a certain distance.

"It is supposed that the normal velocity is sufficient, and no more, to carry to the sea the sediment brought to the alluvial portion from above. This will not prevent it from acting on the banks. Be the water never so highly charged, provided its fluidity be not materially impaired, when it encounters the bank at an abrupt angle, it will not fail to dislodge a portion of the material. This action is purely mechanical. It is the effect of direct impact, like the blow of a pickaxe or a hydraulic jet. The earthy matter thus committed to the water will not sink directly to the bottom. It will be carried a hundred or a thousand feet, or a mile, or ten miles, and eventually dropped, and this although there were no slackening of the velocity. By hypothesis the water was already loaded with all it could permanently transport, when it acquired this additional load, which must therefore be only temporary, *a fortiori* then must be deposited when the velocity is diminished. The 'caving' action, as it is called, generally goes on for several

miles throughout the bend. The velocity is not immediately checked to a considerable extent, the acceleration due to the fall contending with the resistance of the bank. Each 'pocket' excavated affords an additional point of attack to the water from above; the load of sediment becoming accumulated and partially distributed as the different fillets, etc., come in contact with the bank, or with the fillets next the bank; and from the cross and vertical currents arising from the violent disturbance. Finally a point will be reached when the destructive energy is exhausted, the velocity reduced, the angle of presentation of the bank less, and all the conditions favorable to deposition. The load will be thrown down here, forming a shoal. The cross-section will become shallower, therefore wider, and on the whole (the velocity being now at a minimum) enlarged considerably. The slope will therefore be steepened, and the velocity gradually accelerated, until it again becomes normal or a little more, when the same process will be repeated.

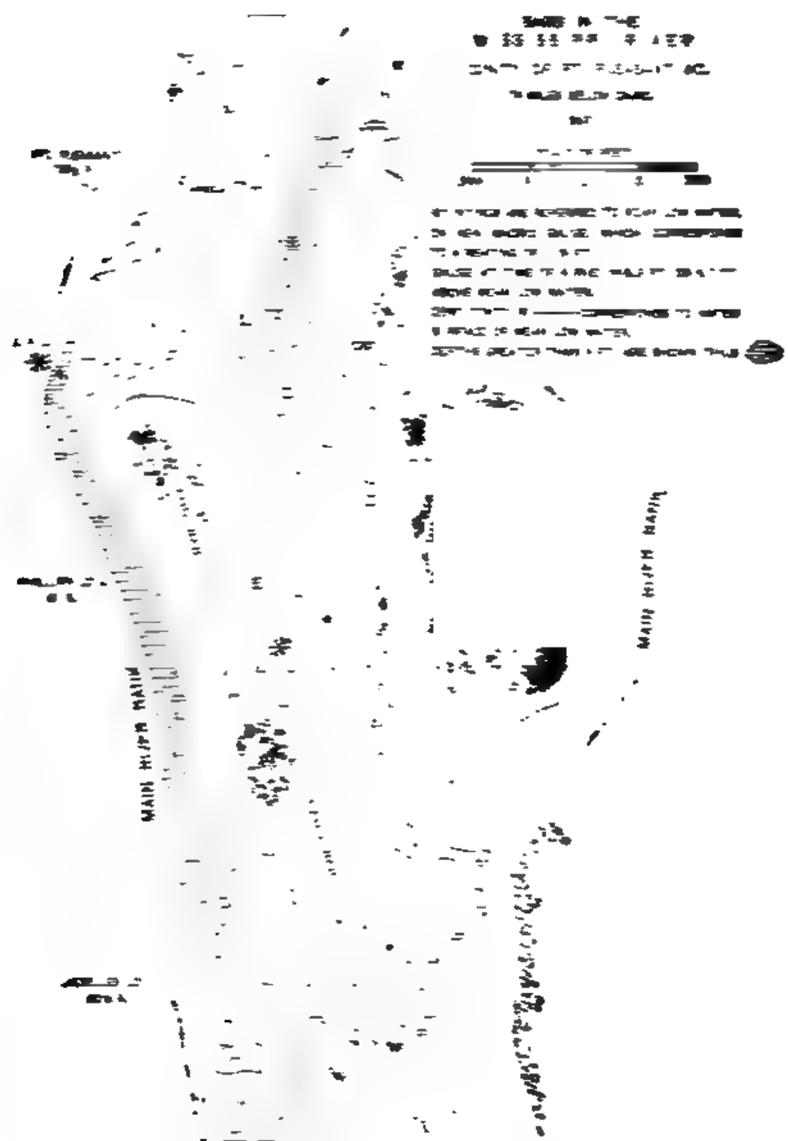
"It is evident, then, that the tendency of the river, if unchecked, would be to go on lengthening the bends. This would gradually give a flatter slope to the whole stream, so flat that it would not be compatible with a velocity sufficient to carry the sediment to the sea. There would, therefore, be a deposit in the upper part of the alluvial portion, and a raising of the flood-plain; and this process would go on indefinitely, assisted by the lengthening of the stream due to the advance of its delta into the sea. A check is given to this development by two agencies.

"*First.*—The eroding itself cannot proceed beyond a limited extent, for it almost ceases when the bank attacked is at less than a certain angle with the current. If water loaded to its normal capacity flow through straight banks, it will dislodge no more. Neither will it at a gentle angle. In short, it is found by observation that the caving process does not proceed to any considerable extent in ordinary soil and with the ordinary high-water velocity when the radius of curvature of the bend exceeds about two miles.

"*Second.*—The river thus works out for itself a bend, which would be circular if the material were homogeneous and the velocity continued unchanged. It has just been explained why it is not immediately diminished below the normal by impact against the bank—viz., on account of the acceleration due to the fall from the shoal above. But even this acceleration is presently overcome, the velocity is lessened, the curve becomes flatter, erosion ceases, and the current leaves the bank which it has so far hugged, with the direction given by the last reach, to cross over to the other side. If this were not so, the river would speedily return upon itself and form a complete circle. As it is, this tendency is only partially overcome. The river does eventually almost return upon itself until the neck becomes exceedingly narrow. Sometimes the overpour during high water across the narrowest part of the neck forms a deep gorge on the lower side, which cuts back year by year toward the upper side, after the ordinary manner of cataracts. During some great flood the velocity of the water, now precipitated across the neck with a fall of perhaps 4 or 5 feet in a thousand, becomes irresistible, and opens a

passage for the whole river, thus forming what is called a cut-off. See Fig. [illegible], page [illegible], and Fig. [illegible], page [illegible].

* There is another cause which favors the formation of cut-offs. On account of the




[illegible] of the bank and bottom in concave bends, the water becomes very deep, and cross-section is formed so favorable to the movement of water, that a comparatively small slope is sufficient to give it the velocity requisite. This velocity is at length partially exhausted by the resistance of the curved bank, until finally, in the course of a long and deep bend the river, even in flood-time, is approximated to the condition of a pool. Hence there is a tendency to concentrate the fall into the reaches

above and below the pool, so that the slope is alternately greater, less and greater than the mean. Thus the disposition to erosion at both the upper and lower sides of the neck is increased.

"So, also, the formation of a quasi-pool in the bend creates a tendency in the water to seek more direct routes. As the river pilots express it, 'When a bend becomes very round, the water wants to leave it.'

"Now, almost all 'points' have been formed by successive rudimentary islands, which afterwards grow into perfect islands, become covered with vegetation, and eventually are raised to nearly mean flood-mark. These islands are divided from the main land by channels called 'chutes,' which are never entirely obliterated, but generally serve as high-water conduits. Frequently, in a bend which has grown very 'round,' the river will seek such a short cut and make it the main channel, thereby shortening its course several miles.

"It should be observed that these changes do not occur periodically, or in obedience to any rational demand, so to speak, for compensation. They are local phenomena. Sometimes they happen unseasonably, in advance of any undue lengthening of the river, or when it has already been shortened by other cut-offs. In fact, it has been observed that they frequently occur in cycles, several in succession, where a number of 'points' exist in close proximity, presenting favorable conditions for cut-offs.

"Lest the river should become unduly short by these means, it proceeds to lengthen itself with great rapidity, by increased erosion of the bends and building out of the points in the vicinity of the cut-off.

"In these ways the disposition of the river to keep on increasing in length is compensated, and the total mean distance from the head of the alluvial basin to the beginning of the delta preserved nearly unaltered, subject only to oscillations. The Mississippi has been observed with accuracy for only a few years, but there is no evidence of any material general elevation of the bed, or lengthening of the course, since attention has been directed to it."

**Characteristics of Indian Rivers.**—A brief description of the characteristics of the alluvial portions of certain rivers of Northern India, such as the Indus and the Ganges, will also be of interest as exhibiting the universalness of application of the laws governing these phenomena. For lack of stability of bed and rapidity of erosion these rivers are probably unsurpassed.*

"The Mississippi and the Lower Ganges are probably more alike in the characteristics that are of interest to their engineers than are, say, the Kistna and the Brahmaputra, the Godavari and the Indus, of the Narbada and the Ganges. The high and well-defined bluffs which often mark the outer limits of the Mississippi's meanderings would appear to be more pronounced than the often scarcely discoverable 'permanent banks' (we do not say 'bluffs') which demarcate the *khádirs*, or flooded bottoms, of our

* "River Training and Control on the Guide Bank System," Francis J. E. Spring.

Indian rivers. Perhaps the 15-mile stretch of '*Barind*' along the left bank of the Lower Ganges, between Godagari and Rampur Boalia, or the high banks of the Indus below Kalabagh, or the bluffs demarcating the edges of the high *bár* of some of the Punjab *doábs*, may bear some resemblance to the Mississippi bluffs. But, as a rule, there is little in the composition of the 'permanent banks,' at the outer edges of the *khádirs* of Indian alluvial rivers, which would enable them to stand up against serious erosion. In other words, the bluffs of our Indian rivers may, as a rule, be regarded as the limits within which the rivers may have seen fit to meander, rather than as barriers against their wider meanderings.

"The alluvial rivers of Northern India flow for the most part through what, to the unaided eye, seems like an unbroken plain devoid of anything that can properly be called a valley. The making of a regular contour survey, however, proves the existence of a certain amount of slope, which along the river axis may run to from say 3 feet per mile near the hills to 3 inches per mile near the sea. The cold weather—November to June—discharge of these rivers is usually very small, and in many cases most of what there is of it is carried off for irrigation. Steamers, however, of 2 to 6 feet draft—nowadays driven off, to a great extent, by the railways—and great native craft, the latter in countless numbers and up to 100 tons burden, ply on most of the rivers all through the cold months, so long as wind and channels permit. The Himalayan snows begin to melt in May, and the rivers of the class under notice usually begin to fill up early in June, and to stand in flood, at anything from 8 to 30 feet above low-water level, until October. The floods of some of them are affected greatly by local rain, those of others are so affected but slightly.

"If it be permissible to generalize so far, in order to give a fair idea of the comparative size of these rivers to engineers elsewhere who have not seen them, it may be said, in a general way, that cold weather discharge may vary from 1000 to 20,000 cubic feet per second, while flood discharge may be anything from, say, 100,000 to 2,000,000 feet per second, according to the river. The cold weather channel may be a mere thread, say 100 to 300 feet wide, or it may be 1000 to 3000 feet wide on the larger rivers, with velocities from *nil* to, say, 4 feet per second. The flood channel may be anything from half a mile to one mile wide in the minor rivers, or up to 10 to 12 miles wide in the larger rivers. Where these great breadths of flood occur, the whole is, of course, not flowing with anything like an uniform velocity; for parts may scarcely be stemmed by a ten-mile-per-hour steamboat, while miles in breadth of shallow flow may not be moving at a faster rate than a half foot per second. The Indus at 700 miles from the sea may have to be crossed any year, in August or September, by 10 or 12 miles of a ferry. Similarly the Ganges or the Brahmaputra at 300 miles from the sea may be 5 to 10 miles wide. Practically, none of those rivers is fordable within 600 miles from the sea except under rare circumstances.

" It is at the height of the monsoon that the magnitude of the floods is properly seen. Then the banks, which may be 60 feet high, are full to within a few feet of the top, with a great sea of turbid water. Viewed from the summit of what were in summer high cliffs, the river runs just beneath one's feet. Far out in the center of the current a line of floating objects goes along. Great trees torn up by the roots sail steadily, their heads and branches, limbs and roots rolling alternately above the raging torrent. Houses and logs, the bodies of cattle and deer, and even human remains, float away in constant procession carried by the relentless force of the waters on their voyage of many hundred miles to the ocean. The sky is lurid with heavy rain, charged clouds, at times discharging torrents of drenching warm rain. At times the thunder rolls, and flash after flash of lightning rends the sky, and the surface of the torrent is whitened and lashed by the howling, hot tempest.

" The manner in which such rivers find it necessary to meander more and more the nearer they get towards the sea—in other words the lighter and less coherent becomes the sand composing their beds—may be illustrated by a rough measurement of the Indus between Kalabagh, where it first becomes alluvial, and the sea. In the following table, column A gives rough measurements along the general lie of the river, while column B gives more accurate measurements round the bends. Both columns have been scaled off a 32-mile to the inch map.

| Successive 100-mile Lengths. | *A* Length Measured Fairly Direct. | *B* Length Measured Round Bends. | Percentage of Meandering. |
|---|---|---|---|
| First 100 miles, beginning at the sea | 72 | 100 | 39 |
| Second 100 miles, " " | 75 | 100 | 33 |
| Third 100 miles, " " | 72 | 100 | 39 |
| Fourth 100 miles, " " | 69 | 100 | 45 |
| Fifth 100 miles, " " | 82 | 100 | 22 |
| Sixth 100 miles, " " | 82 | 100 | 22 |
| Seventh 100 miles, " " | 93 | 100 | 7 |
| Eighth 100 miles, " " | 98 | 100 | 2 |
| Ninth 100 miles, to near Kalabagh | 97 | 100 | 3 |

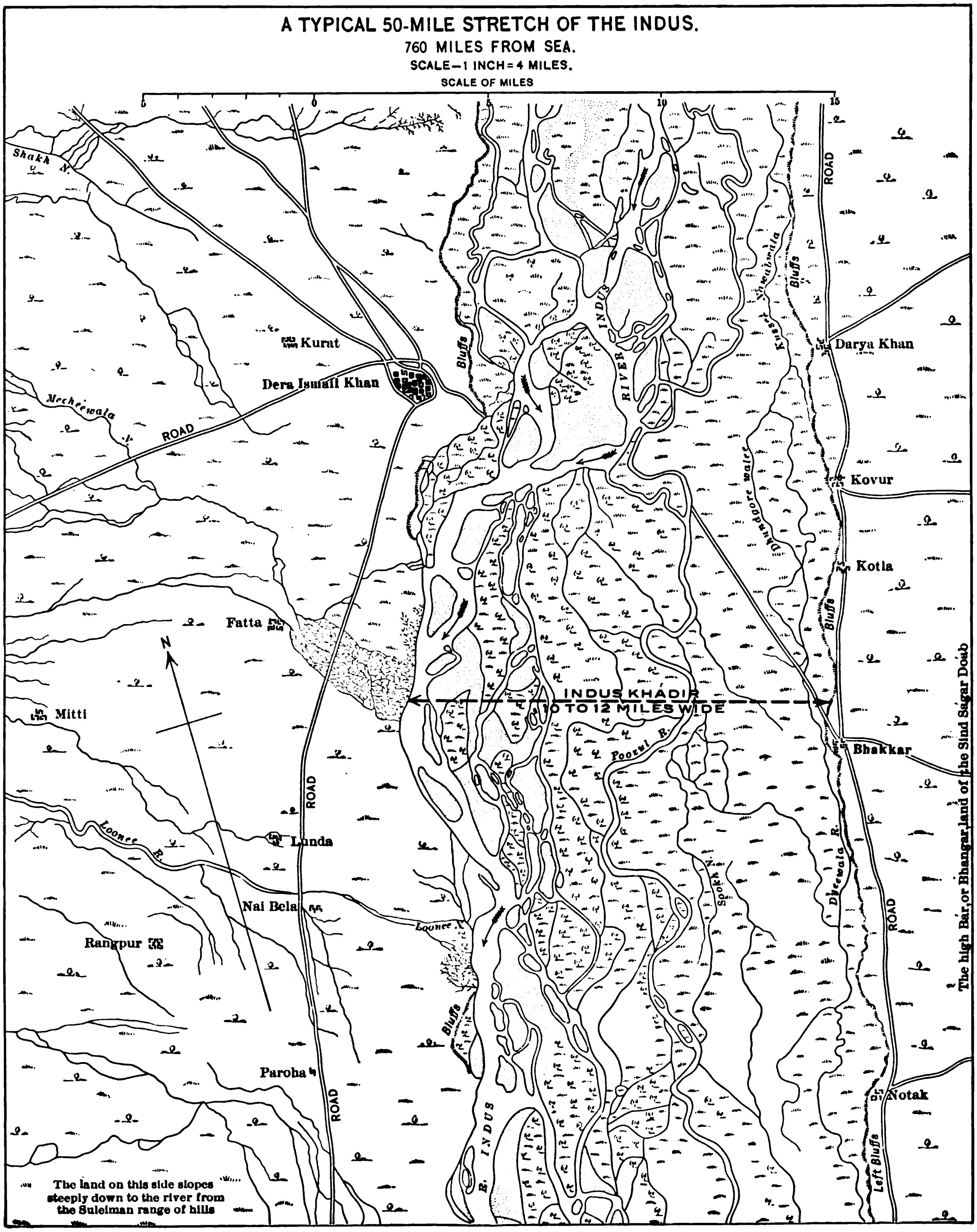
" But even when such rivers as are being described have meandered as far as they ever will within the limits of the shallow valleys which they have scooped out for themselves in the alluvium, they are no nearer than ever to the attainment of a state of permanency or equilibrium within those limits. Indeed, in the course of years there is scarcely an acre within the valley limits which will not sooner or later be eroded quite away, and in turn re-formed. Action of this sort goes on for the most part during flood time, and much of it beneath the surface of the silt-laden water. What happens is somewhat as follows: When the flood begins to rise with the melting of the Himalayan snows in June, it finds, (*a*), ready prepared for it since the previous year, a more or less tortuous and more or less deep channel. As it rises higher (*b*) it spreads itself over the low sandy spits and fills short-cut channels. At its half-flood

stage (*c*) it finds itself topping extensive areas covered with heavy reeds, grass and brushwood. Later again (*d*), if it should happen to rise somewhat higher than in recent years, it tops cultivated areas and drives the inhabitants of the more or less temporary villages, to whom the cultivation is due, to take shelter in trees, boats or house tops. This last state of things (*d*) is seldom of more than a few days' duration, but the conditions described at (*b*) may be of six months' and the intermediate stage (*c*) of perhaps three or four weeks' duration.

"Now, owing to the comparatively brief time during which the flood tops the higher levels where cultivation is being carried on, it seldom has time to do any surface erosion at such places; on the contrary, the tendency is to raise such places by the deposition of silt, owing to the great reduction of velocity due to the lessened depth of water and the restraining influence of the vegetation. Moreover, a skin of vegetation, though of no avail against edge, or caving, erosion, is a very effective preventive of surface erosion. Therefore, so far as mere superficial action is concerned, the tendency is for the higher places, below highest flood level, to grow higher still, to the great profit of the cultivators, to whom a few days of exposure and semi-starvation are of slight importance in comparison with the increased fertility of their lately flooded fields.

"But, flowing as it does over a great breadth of valley, swiftly in the deeper parts where last season's channel is, and more slowly in the shallower parts, the river finds itself taking shallow short-cuts across great bends, at a comparatively low velocity owing to the shallowness, and then cataracting down into the main stream at a very high local velocity at the downstream end of each short-cut.* It often happens, if the bend is a very big one, that the short-cut channel has not time to erode its bed and banks to any great extent; and in this case, on the fall of the river, the results of the action will be exhibited in the form of an abortive channel which, had the flood only lasted a few days or hours longer, might have established itself right across the bend. But should the flood stand up long enough for the short-cut channel to erode itself adequately we find the following state of things, viz., (*a*) the main river running round a great curve, say 10 miles long, with the velocity due to a fall of, say, 1 foot per mile measured round the bend, and with a favorable stream cross-section; and (*b*) a comparatively small side chute, with the same fall, say 10 feet, in perhaps 3 miles instead of 10, but handicapped by its comparatively unfavorable cross-section. If the net result of the favorable fall and unfavorable section should be that the velocity in the narrow short-cut is effective in cutting its way so as rapidly to widen and deepen the small side channel or chute, the consequence will be that, in a few days or hours—the author has seen such a change occur on a very large scale in twenty-four hours—the small side channel will have constituted itself the main river, widening itself by caving, and the long bend will have silted up.

* The banks of such rivers are low, and the side channels (see accompanying map of the Indus) may fill at quarter or half flood height. Hence a cut-off may occur very easily.


A TYPICAL 50-MILE STRETCH OF THE INDUS.
760 MILES FROM SEA.
SCALE—1 INCH = 4 MILES.
SCALE OF MILES
5
0
5
10
15
Shakh N.
Kurat
Dera Ismail Khan
Mecheewala
ROAD
Fatta
Mitti
N
Loonee R.
Lunda
Nai Bela
Loonee N.
Rangpur
Paroha
ROAD
Bluffs
INDUS
RIVER
INDUS KHADIR
10 TO 12 MILES WIDE
Poozul R.
Sookh N.
Deewala R.
Dundgore water
Bluffs
Darya Khan
Kovur
Kotla
Bhakkar
Notak
Left Bluffs
ROAD
B. INDUS
The high Bar, or Bhangar land of the Sind Sagar Doab
The land on this side slopes steeply down to the river from the Suleiman range of hills

stage (*c*) it finds itself topping extensive areas covered with heavy reeds, grass and brushwood. Later again (*d*), if it should happen to rise somewhat higher than in recent years, it tops cultivated areas and drives the inhabitants of the more or less temporary villages, to whom the cultivation is due, to take shelter in trees, boats or house tops. This last state of things (*d*) is seldom of more than a few days' duration, but the conditions described at (*b*) may be of six months' and the intermediate stage (*c*) of perhaps three or four weeks' duration.

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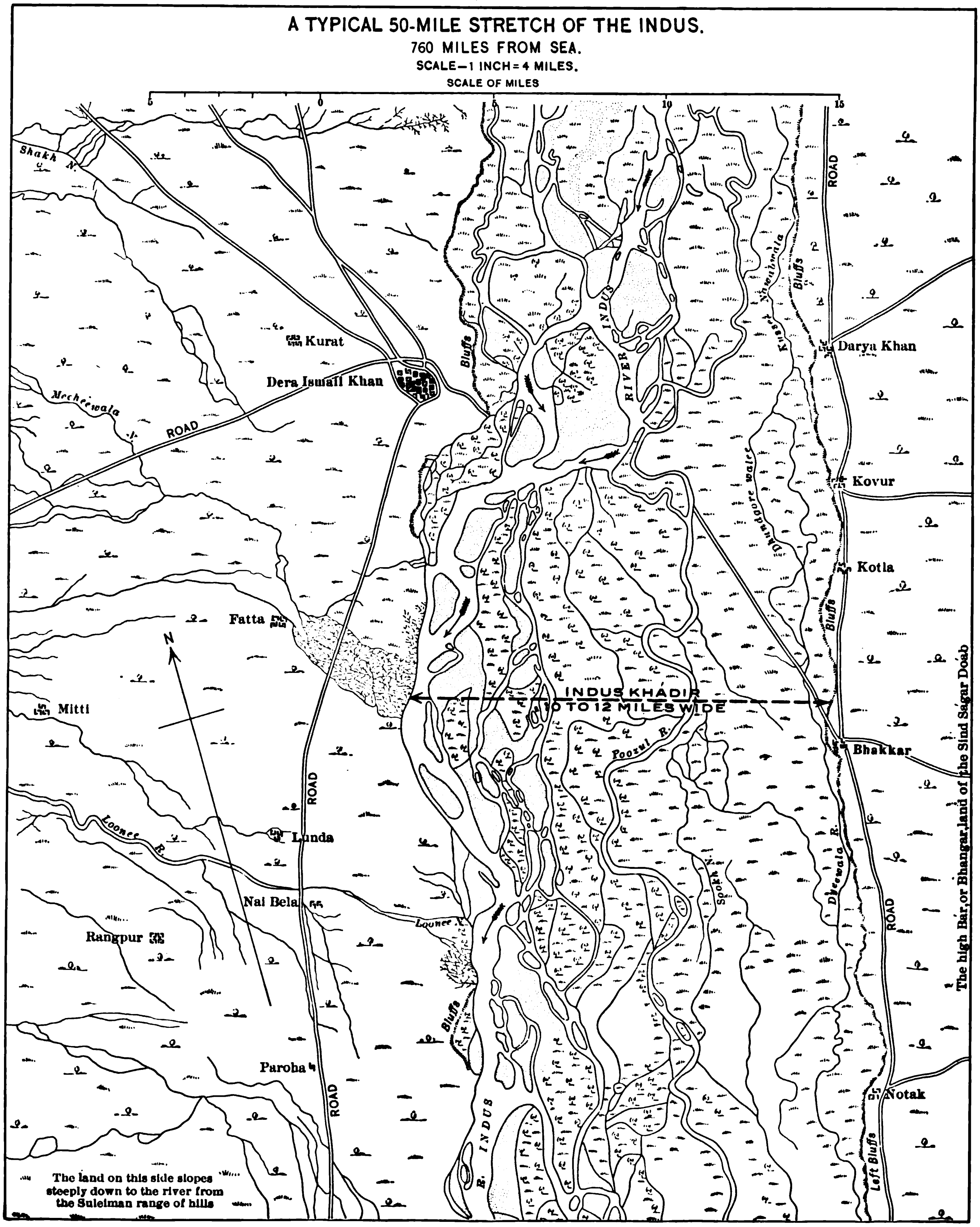
A TYPICAL 50-MILE STRETCH OF THE INDUS.
760 MILES FROM SEA.
SCALE—1 INCH=4 MILES.
SCALE OF MILES
5
0
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15
Shakh N.
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Dera Ismail Khan
Mecheewala
ROAD
Fatta
N
Mitti
ROAD
Loonee R.
Lunda
Nai Bela
Loonee N.
Rangpur
Paroha
ROAD
The land on this side slopes steeply down to the river from the Suleiman range of hills
Bluffs
RIVER INDUS
INDUS KHADIR 10 TO 12 MILES WIDE
Poozul R.
Spokh N.
Bluffs
B. INDUS
Dhundgore water
Kuser Nuwuhwala
Bluffs
ROAD
Darya Khan
Kovur
Kotla
Bluffs
Bhakkar
Dheewala R.
ROAD
Notak
Left Bluffs
The high Bar, or Bhangar, land of the Sind Sagar Doab

first thing the current will do is to drop its coarser grains of sand; equally naturally, on regaining its original velocity by passing again into a deep place, it will, by preference, pick up the finer particles with which it comes in contact, in lieu of the coarser ones. The net result is that the finer particles have a better chance in the long run—and it must be realized that 'in the long run' may mean many thousands of years—of getting on towards the sea, and that the delta part of the country, raised in the course of ages out of the sea by the river's depositions, usually consists of almost impalpable matter. For as each individual grain of matter approaches the sea in its journey of years or centuries, the chances are in favor of its getting crushed or worn or broken smaller and smaller, and so of its not, after all, being left behind in the race."

DISCHARGES OF VARIOUS RIVERS*

| River and Country. | Discharge in Cubic Feet per Second.† | | | Ratio of Min. to Max. | Extreme Range between High and Low Water. | Remarks. |
|---|---|---|---|---|---|---|
| | Minimum. | Maximum. | Annual Mean. | | | |
| *North America* | | | | | Feet. | |
| Columbia | 48,500 | 1,390,000 | 67,000 | 1 to 28.7 | .......... | Measured at the Dalles; comprises 99% of the drainage. Mean is for 30 years. |
| Mississippi at St. Paul | 1,500 | 117,000 | .......... | 1 to 23.4 | 19.7 | Average low-water dis- at St. Paul is 2500 sec.ft. |
| Mississippi at St. Louis | 30,000 | 1,200,000 | 225,000 | 1 to 40.0 | 42.5 | Flood height given for 1844. |
| Mississippi at Cairo | 45,000 | 1,507,000 | .......... | 1 to 33.5 | 54.0 | Flood height given for 1912; discharge for 1893. |
| Mississippi at Vicksburg | .......... | 1,617,000 | .......... | .......... | 52.5 | Flood height given for 1897; discharge for 1903. |
| Mississippi at New Orleans | 65,000 | 1,740,000 | .......... | 1 to 26.8 | 21.4 | Flood height given for 1912. |
| Missouri at Sioux City | 7,000 | 650,000 | .......... | 1 to 92.8 | 19.0 | |
| Missouri at mouth | 15,000 | 900,000 | 100,000 | 1 to 60.0 | 35.0 | |
| Niagara | 175,000 | 260,000 | 219,850 | 1 to 1.49 | .......... | |
| Ohio at Pittsburgh | 1,400 | 439,000 | .......... | 1 to 313.5 | 35.6 | Flood of 1907. |
| Ohio, just below Kanawha River | 5,600 | .......... | .......... | .......... | 63.5 | Flood of 1884 at Cincinnati rose nearly 71 ft. above low water. |
| Ohio, just below Kentucky River | 6,900 | .......... | .......... | .......... | 59.4 | |
| Ohio, at mouth | .......... | .......... | .......... | .......... | 54.0 | Flood of 1912. |
| Ottawa, at mouth | 17,400 | 250,000 | .......... | 1 to 14.3 | .......... | |
| Rio Grande, at El Paso | .......... | 16,600 | 1,500 | .......... | .......... | In dry seasons the flow is all sub-surface. |
| St. Lawrence | 185,000 | 330,000 | 251,900 | 1 to 1.78 | .......... | |
| Sacramento, at Collinsville | 5,050 | 160,000 | 37,600 | 1 to 31.6 | .......... | |
| Tennessee, at Chattanooga | 3,700 | 468,000 | .......... | 1 to 126 | 58.6 | |
| Tenneessee, at Riverton, Ala. | 10,000 | .......... | .......... | .......... | .......... | |
| *Europe* | | | | | | |
| Danube, in Upper Hungary | 24,000 | 380,000 | .......... | 1 to 15.8 | .......... | |
| Danube, at head of Delta | 70,000 | 1,000,000 | .......... | 1 to 14.3 | .......... | |
| Dnieper, at the cataracts | 9,800 | 305,000 | .......... | 1 to 31.2 | .......... | |
| Elbe, in Bohemia | 1,350 | .......... | 9,000 | | .......... | |

* These tables have been compiled from Government reports, various engineering publications, etc. Owing to the many sources, the names of the authorities have been omitted.

† The abbreviation "second-feet" which is frequently used in America instead of "cubic feet per second," has the equivalent of "cusecs" among the Anglo-Indian engineers.

DISCHARGES OF VARIOUS RIVERS—*Continued*

| River and Country. | Discharge in Cubic Feet per Second. | | | Ratio of Min. to Max. | Extreme Range between High and Low Water. | Remarks. |
|---|---|---|---|---|---|---|
| | Minimum. | Maximum. | Annual Mean. | | | |
| *Europe* | | | | | Feet. | |
| Elbe, at Hamburg | 15,300 | 120,000 | 53,300 | 1 to 7.8 | .......... | |
| Garonne, at Toulouse | 1,270 | 212,000 | .......... | 1 to 167 | 33.0 | |
| Loire, at Briare | 1,240 | 322,000 | .......... | 1 to 258 | .......... | Flood of 1846. Flood range at Amboise, 27 ft. |
| Meuse, at Sedan | 450 | 24,700 | .......... | 1 to 55 | .......... | Flood of 1846; low water of 1893. |
| Moldau, at Prague | 706 | 141,000 | 2,410 | 1 to 200 | .......... | |
| Moselle, at Liverdun | 282 | 39,000 | .......... | 1 to 138 | .......... | |
| Oder | 120 upper end<br>6700 lowerend | .......... | .......... | .......... | .......... | |
| Po (about 60 miles above mouth) | .......... | 221,000 | 60,700 | .......... | .......... | |
| Rhine, at Strasburg | 13,500 | .......... | .......... | .......... | .......... | |
| Rhine, at Coblenz | 20,000 | .......... | .......... | .......... | .......... | Flood range at Colonge, 29 ft. |
| Rhine, at Emmerich | 26,800 | 353,000 | .......... | 1 to 13 | .......... | Guillemain gives the annual mean for the Rhine as 39.200 sec.-ft.; other observers as 70,000 ft. between Bingen and Emmerich. |
| Rhone, below mouth of Saône | 5,300 | 247,000 | .......... | 1 to 46 | 21.0 | Flood of 1856; low water of 1884. |
| Rhone, below mouth of Durance | 13,000 | 490,000 | .......... | 1 to 38 | .......... | Flood of 1856; low water of 1884. |
| Saône, at Chalons | 1,400 | 106,000 | .......... | 1 to 76 | 34.0 | Flood of 1840. |
| Seine, at Paris | 1,700 | 88,000 | 8,700 | 1 to 51.8 | 29.0 | Flood of 1910; low water of 1858. |
| Theiss | 2,100 | 131,000 | .......... | 1 to 62.5 | .......... | |
| Tiber, at Rome | 3,500 | 140,000 | .......... | 1 to 40 | .......... | |
| Vistula, at mouth | 8,770 | 293,000 | .......... | 1 to 33.3 | .......... | |
| Volga, at Rybinsk | 7,000 | 223,000 | .......... | 1 to 32 | 42.0 | At Tver, on the upper portion of the river, the low-water discharge is 3900 sec.-ft., and the flood range is about 37 ft. |
| Volga, above mouth of Kama | 52,000 | 350,000 | .......... | 1 to 6.7 | 50.0 | |
| Volga, at head of delta | 113,000 | 1,426,000 | .......... | 1 to 12.6 | 12.0 | Flood of 1884. |
| *Other Countries* | | | | | | |
| Lower Ganges, near Sara | .......... | 2,050,000 | .......... | .......... | 48.0 | Guillemain gives the annual mean for the Ganges as 532,000 cu.ft. |
| Kistna * | 100 | 770,000 | .......... | 1 to 7,700 | .......... | |
| Godavari * | 50 | 1,200,000 | .......... | 1 to 25,000 | .......... | |
| Indus | 18,000 | 800,000 | | 1 to 44.4 | 8 to 10 | |
| Nile, at Assuan | 12,000 | 465,000 | 107,000 | 1 to 38.8 | 32.5 | Discharge is less at Cairo because of loss from evaporation and irrigation. The Delta canals take about one-fifth of the average annual discharge below Cairo. |
| Nile, at Cairo | 8,800 | 425,000 | 93,000 | 1 to 48.5 | 31.6 | |
| Paranà, near mouth | .......... | 1,100,000 | 765,000 | .......... | 24.0 | This flood discharge is for the main river, and does not include any flow over submerged lands. |

* The rivers of southern India, being dependent solely on rains for their supply of water, have an enormous variation of discharge; those of northern India are less variable, being fed in the summer months from the melting of the Himalayan snows.

SLOPES AND SURFACE VELOCITIES.

| River. | Section. | Length of Section. Miles. | Total Fall in Low Water. Ft. | Slopes in Feet per Mile. Minimum. | Maximum. | Average. | Surface Velocities, Feet per Second. Feet. | Remarks. |
|---|---|---|---|---|---|---|---|---|
| Amazon... | Tomependa to mouth.... | 2500 | .... | .... | .... | 0.5 | ......... | |
| Danube... | Donauschingen to Vienna | 680 | .... | .... | .... | 2.6 | ........ | |
| " ... | Vienna to mouth....... | 1060 | .... | .... | .... | 0.48 | ........ | The average fall from Vienna to the mouth, 1060 miles, is 0.48 ft. per mile. |
| Elbe...... | ..................... | .... | .... | 0.35 | 2.06 | .... | 1.5 to 3.4 | At nav. stages. |
| Ganges.... | Lower 300 miles........ | 300 | .... | .... | .... | 0.25 | ......... | |
| Garonne... | Toulouse to the Tarn..... | 51 | ... | 0.2 | 6.3 | 4.1 | ......... | |
| " ... | Tarn to Bordeaux....... | 126 | ... | | | 1.6 | ......... | |
| Loire..... | Roanne to Orleans...... | 202 | ... | 0.9 | 3.7 | 2.9 | ......... | |
| " ..... | Orleans to mouth....... | 218 | ... | | | 1.4 | ......... | |
| Meuse.... | Pagney to Belgium...... | 220 | .... | .... | .... | 0.2 | ......... | |
| Mississippi | St. Paul to St. Louis..... | 676 | 305.8 | 0.01 | 2.41 | 0.45 | ......... | |
| " | St. Louis to Cairo ...... | 180 | 108.4 | 0.22 | 0.9 | 0.60 | ........ | The total fall from Cairo to the sea is about 268 ft., an average of 0.25 ft. per mile for the 1073 miles. The low-water slope varies from 1.14 ft. per mile to almost zero, and the high-water slope from from 0.76 ft. to about 0.10 ft. |
| " | Cairo to St. Francis River | 298 | .... | .... | .... | 0.42 | 7.7 in flood | |
| " | St. Francis River to Yazoo River............ | 297 | .... | .... | .... | 0.33 | ........ | |
| " | Yazoo River to Red River | 170 | .... | .... | .... | 0.27 | 7.6 in flood | |
| " | Red River to New Orleans | 199 | .... | .... | .... | 0.14 | 6.0 to 7.3 in flood | |
| " | New Orleans to head of Passes............... | 96 | .... | .... | .... | 0.10 | ........ | |
| Missouri.. | Sioux City to mouth..... | 807 | .... | 0.70 | 1.12 | 0.86 | ......... | |
| Nile...... | Khartoum to mouth..... | 1870 | 1275.0 | 0.21 | 6.6 | 0.68 | ........ | In flood the maximum velocity over the cataracts is 7.5 ft. per sec., and along the river elsewhere, 5.7 ft. In low water the ordinary velocities are from 2.8 to 3.3 ft. |
| " ...... | Head of delta to mouth... | 78 | .... | 0.25 | 0.42 | 0.41 | ........ | |
| Oder...... | ...................... | .... | .... | 0.25 | 1.84 | .... | 5 to 12 in fl'd | |
| Ohio...... | Pittsburgh to Dam No. 6 | 29 | 42.3 | .... | .... | 1.46 | ........ | |
| " ..... | Dam No. 6 to Marietta.. | 142 | 85.7 | .... | .... | 0.60 | ........ | |
| " ...... | Marietta to Cincinnati... | 296 | 138.1 | .... | .... | 0.47 | ........ | |
| " ...... | Cincinnati to Louisville.. | 133 | 27.2 | .... | .... | 0.21 | ......... | |
| Po....... | ..................... | .... | .... | 0.3 | 2.1 | .... | 5 to 6.5 | |
| Rhine..... | Basle to mouth......... | 537 | 806.0 | 0.63 | 10.1 | 1.5 | 4.3 to 13.0 | |
| " .... | Strassburg to Holland... | 355 | .... | 0.63 | 3.1 | 1.17 | 4.3 to 10.0 | |
| Rhone.... | Lake Geneva to Perrache | .... | .... | .... | .... | 4.1 | ......... | |
| " .... | Perrache to Arles....... | .... | .... | .... | .... | 2.9 | ......... | |
| " .... | Arles to mouth......... | .... | .... | .... | .... | 0.2 | ......... | |
| " .... | Lyons to mouth......... | 206 | 520 | 1.6 | 4½–10 | 2.5 | 3.3 to 13.0 | |
| Saône..... | Gray to Châlons........ | 83 | ... | 0.2 | 1.2 | 0.6 | ......... | |
| " ..... | Châlons to Lyons....... | 87 | ... | | | 0.5 | ......... | |
| Seine..... | Montereau to Paris...... | 64 | .... | .... | .... | 1.2 | ......... | |
| " ..... | Paris to mouth......... | 230 | .... | .... | .... | 0.5 | ......... | |
| Tennessee. | Chattanooga to Riverton. | 238 | .... | .... | .... | 0.4 to 2.7 | ......... | |
| " | Riverton to mouth...... | 226 | .... | .... | .... | 0.34 | ......... | |
| Vistula.... | ..................... | .... | .... | 0.28 | 0.93 | | | |
| Volga..... | Rybinsk to Nidjni Novgorod................. | 296 | 70 | .... | .... | 0.23 | 0.8 to 7.3 | |
| " ..... | Nidjni Novgorod to mouth of Kama....... | 297 | 84 | .... | .... | 0.28 | | |
| " ..... | Kama to head of delta... (Astrakhan).......... | 1060 | 165 | .... | .... | 0.15 | | |
| " ..... | Rybinsk to head of delta. | 1653 | 319 | .... | .... | 0.19 | ......... | |
| " ..... | Source to head of delta... | 2254 | 701.5 | .... | .... | 0.31 | ......... | |

SEDIMENT TRANSPORTED

| River. | Grammes per Cubic Meter. | | | Grains per Gallon. | | | Total Cubic Yards Discharged per Annum. | Remarks. |
|---|---|---|---|---|---|---|---|---|
| | Min. | Max. | Annual Mean. | Min. | Max. | Annual Mean. | | |
| Arkansas | .... | .... | ..... | .... | .... | .... | 26,000,000 | See also page 5. |
| Brahmaputra | .... | .... | 3000 | .... | .... | 175 | .......... | |
| Danube | 100 | 1100 | 283 | 6 | 64 | 16 | 78,500,000 | |
| Ganges | .... | .... | 1493 | .... | .... | 87 | .......... | |
| Garonne | .... | .... | ..... | .... | .... | .... | 4,500,000 | |
| Kistna | .... | 2290 | ..... | .... | 134 | | .......... | |
| Mississippi (Cairo to Gulf) | 200 | 2560 | 800 | 11 | 150 | 46 | 518,500,000 | |
| Missouri | 530 | 5640 | ..... | 31 | 330 | .... | 413,000,000 | |
| Nile | 48 | 1500 | 313 | 3 | 87 | 18 | 47,800,000 | The total cubic yardage given is the amount discharged into the Mediterranean; a large additional amount is carried into the irrigation canals. |
| Paranà at Sta. Fé | 53 | 233 | 100 | 3 | 14 | 6 | 45,000,000 | |
| Paranà at mouth | 42 | 179 | | 2 | 10 | | | |
| Po | .... | .... | 1380 | .... | .... | 80 | 52,000,000 | Vernon-Harcourt gives 35,000,000. |
| Rhone | .... | .... | 870 | .... | .... | 51 | 27,500,000 | |
| Solo (Java) | .... | 5980 | 2290 | .... | 350 | 134 | .......... | |

The following are the ratios between the customary units used in measuring sediment:
1 grain per gallon = 7.47 grains per cu.ft. = 17.1 grammes per cu. meter = 1 part in 58,300 by weight (avoirdupois).
1 grain per cu.ft. = 0.134 grain per gallon; 1 gramme per cu. meter = 0.058 grain per gallon; 1 cu. meter = 35.32 cu.ft. = 1.31 cu.yds.; 1 cu. ft. = 0.028 cu. meter.
1 cu.ft. = 7.47 gallons; 1 ton = 1 cu.yd. for dry silt = $\frac{2}{3}$ cu.yd. for wet silt.

## CHAPTER II.

### REGULATION.

**Requirements of Improvement.** — The general conditions to be fulfilled for the improvement of navigation in "open" or non-canalized rivers are:

1. A sufficient depth for navigation at all seasons.

2. A width of channel sufficient for safe passage of all craft traveling in opposite directions or in the same direction.

3. The works necessary to secure such results must not be of an obstructing character, nor must they cause the formation of obstacles to navigation, nor lower the water-level sufficiently to create other difficulties.

4. The cost of construction and maintenance must not be out of proportion to the benefits to be derived.

The various methods employed with these ends in view may be generalized under the head of Regulation.

**General Principles.**—Regulation, or regularization as it is sometimes called, has been employed in many countries and to a considerable extent. Its comparatively low cost and the greater rapidity with which the works can be put into execution have recommended it for rivers of great length and fall, where canalization would have been out of the question on account of the expense and the length of time required to construct the works, so that hundreds of miles of river have been rendered navigable where, without this system, commerce would have been very uncertain and perhaps impracticable. In all rivers which are used for navigation the passage of boats during seasons of low water is liable to be hindered at some portions of their course by insufficient depths, sharp or uneven curves, or similar natural obstructions. The channel above and below such places may possess ample facilities for easy navigation, but unless the obstruction can be passed without undue effort, navigation may be seriously hampered and perhaps stopped. On the other hand, in times of flood the depths are ample, but then the river may cut into and in many cases overflows its banks, tending to destroy property and to alter the channel. Regulation may thus have one or both of two aims; the first, to regulate or train the river in low water so that the channel may be available as far as possible for commerce; the second, to prevent the erosion and the overflow of the banks, and assist by this means the securing of a more stable bed. It is the object of this chapter to indicate in general terms the principles usually followed in training a river in the interest of navigation; the methods and the effects of the works will be described in detail in the chapters on " Dredging " and " Dikes"; and the related problems and

methods of procedure involved in guarding against the effects of floods will be treated in the chapters on "Protection of Banks" and "Levees." The subjects, divided for convenience into these general headings, are of necessity closely related, and the matter in any one chapter is more or less dependent upon the others for an intelligent understanding of its various phases.

The general regimen of the flow of rivers, set forth at length in Chapter I, may be summarized from a description of the Garonne by Inspector-General Fargue. He observed the following: (See also p. 78 and Fig. 2, p. 14.)

(*a*) The channel followed the concave bank.

(*b*) The shallows occurred along the convex bank.

(*c*) The sharper the curvature the deeper was the channel and the more projecting the shallow point.

(*d*) The maximum or minimum of the degree of curvature corresponded respectively to the maximum and minimum of depth. This correspondence did not occur, however, exactly at the same point, the deepest place and the greatest projection of the point being downstream of the point of the sharpest curvature.

(*e*) The least depth was below those localities where the concavity changed to convexity, that is, where the channel crossed from one bank to the other. The channel was regular in its longitudinal profile only when the curvature of the axis of flow varied in a gradual and continuous manner, and every abrupt change of curvature was accompanied by an abrupt change of depth.

These relations were found only in those portions of the river where the length of bends, that is, the distance between two consecutive points of inflection, or crossings, was appropriate to the characteristics of the river. Where these relations did not exist, the channel was composed of isolated troughs separated one from the other by shoals or bars which soon reformed after they had been removed by dredging.

Using these facts as a basis on which to reason, M. Fargue formulated the following rules. (See Fig. 9, also Fig. 65, Chapter IV.)

1. In order that the channel may be stable and permanent, it is necessary that each bank present a succession of curves, alternately concave and convex, connected by straight lines.

2. In order that the channel may be deep, it is necessary that the curves be neither too great nor too small.

3. In order that the channel may be regular, it is necessary that the curves be of the transition type, that is, they must commence at the end of a tangent and increase regularly in curvature up to the maximum, and then decrease regularly till they coincide with the next tangent.

Fig. 9.

4. The distance between the banks should vary with two elements, viz., the position and the curvature. Thus between two consecutive points of inflection or crossings the width should vary with the degree of curvature, and should present toward the apex a maximum which should be greater as the degree of curvature of the apex is greater. The width should increase and decrease therefore in such a manner that the bed would be widened toward the apex of the curves and restricted at the crossings where the curvature changes its direction.

5. At these crossings the points of inflection of the two banks should not be opposite each other. The one where the concavity is changed to convexity should be above that where the inverse change is made, at a distance which seems to depend only on the width at the point of inflection.

To the foregoing it may be added that every river displays these same general tendencies of flow, resulting from certain natural laws, and these laws should form the basis of any improvement. Modifications of conditions can be made only within restricted limits, a mathematical determination of which can serve at the best merely as a general guide. The actual results aimed at and the methods adopted should be founded on a close study of related conditions formed by the river itself, bearing always in mind that each locality usually displays some special difference which must be taken into account in planning its improvement.

While these rules form the general basis for planning regulation, it must not be supposed that the construction of artificial bank lines as shown on Fig. 65 will modify the general tendencies described in paragraphs (*a*) to (*e*), p. 51, and which have been described in greater detail on pp. 13 to 18, and elsewhere in Chapter I. The deep water in the bends will still keep along the concave bank, with a shoal along the opposite shore, and any widening of the channel there will be difficult to obtain, so that the construction of works along the convex shore may prove, and often has proved, a useless expense. The principal change, and in many cases the only one to be desired, will take place along the crossing, where the narrowing of the limits of flow will tend to create a deeper and more regular channel. Experience has shown, moreover, that in planning improvements for open navigation the natural regimen of the river should be retained as far as practicable, and that the more this is altered, the more expensive will be the works needed to compel the river to follow the changed conditions. The forces at work in each stream have secured or are tending to secure a condition of general equilibrium between the flow and the bed, but this condition is a dynamic, not a static one; and any change of force will thus tend to produce some corresponding change of bed. The stream itself is a chain of causes and effects which should balance at every part if stability is to exist, and the causes are made up of such incalculable elements—varying discharge, different qualities of soil, alternations of slope, etc.—none of which can be controlled to any extent by the engineer—that the safest course appears to lie in the conservation, as far as possible, of the natural conditions.

The alignment and the action of the forces of a river may be compared to the alignment of a railway and the forces controlling the passage of the trains. The location of a river and of a railway is in each case made up of a series of tangents and curves, suited as far as may be to the localities through which the forces have to move. The sharper the comparative curvature of a river the greater is the local effect upon its banks and bed, and the sharper the curvature of the railway the more energy is required for the passage of the trains and the greater is the tendency to disturb the track. In the latter case, however, compensation can always be made by a reduction of speed, but with a river the velocity is beyond control, and the engineer must plan his works with the knowledge that he is confronting natural laws which cannot be modified even in a slight degree without requiring a correspondingly adequate preparation. For example, the natural location of the river channels is dependent to a considerable extent upon the currents in floods, and these have been given their direction at the locality in question by the conditions of the banks and bed above. As long therefore as the banks and bed remain unchanged—and it is rarely practicable to modify them to any marked extent—there will remain a more or less constant tendency for the river to retain its natural conditions of channel, and the more these are to be departed from, the greater will be the extent and cost of the necessary training works. Changes in the current will also tend to produce corresponding changes in the river just below the works, and these in turn will affect the succeeding banks and bed in proportion to the amount of the alteration. As it is impossible to foretell what the exact effects will be, the works should proceed tentatively and without haste, and the evolving conditions should be watched with care. It is an expensive matter to attempt to remedy any disorganization of a river, and experience and sound engineering both emphatically affirm the need of caution in dealing with its uncertain forces.

The principle of training a river by evolving as far as possible its natural tendencies of flow has been aptly termed by the Dutch engineers, " Regulation by current courses."

**Application.**—It has been mentioned (p. 16) that rivers appear to display, in swinging from side to side as they pass along their course, a principle which gives their currents a definite direction. It was described also how every river appears to be endeavoring to discharge its flow with a minimum of effort, in accordance with the tendency of Nature to accomplish her results with a least waste of labor. Every obstruction, whether a snag, a spur of clay or rock, or an irregularity of channel, produces corresponding disturbances in the flow, which endeavors to undermine or smooth away the obstacle. The river seems to attempt, by such forces as it can control, to alleviate the evil, and thus to establish a cross-section which will permit it to pass along without useless toil. It seems always endeavoring, in other words, to establish the hydraulic mean depth most suitable to its elements, and this mean depth usually represents at the same time the condition of channel most suitable for navigation which the stream can produce. It is by evolving these tendencies therefore—restraining the action where it is too violent

and assisting it where it is too weak—that the engineer may hope to obtain the best results. In the earlier days of river improvement the belief seemed prevalent that as water was a fluid element, without cohesion and influenced apparently only by the law of gravitation, it could therefore be trained almost at will and be made without difficulty to flow in any reasonable direction. The engineers, relying on this supposition, and not having learned as yet the influence upon a river of other important laws, did not hesitate to contract a stream until serious changes resulted in the bed, or to build training dikes by which a current was forced, instead of being led, into a desired channel. It was further believed at that time that the natural condition of a river bed, which consists of

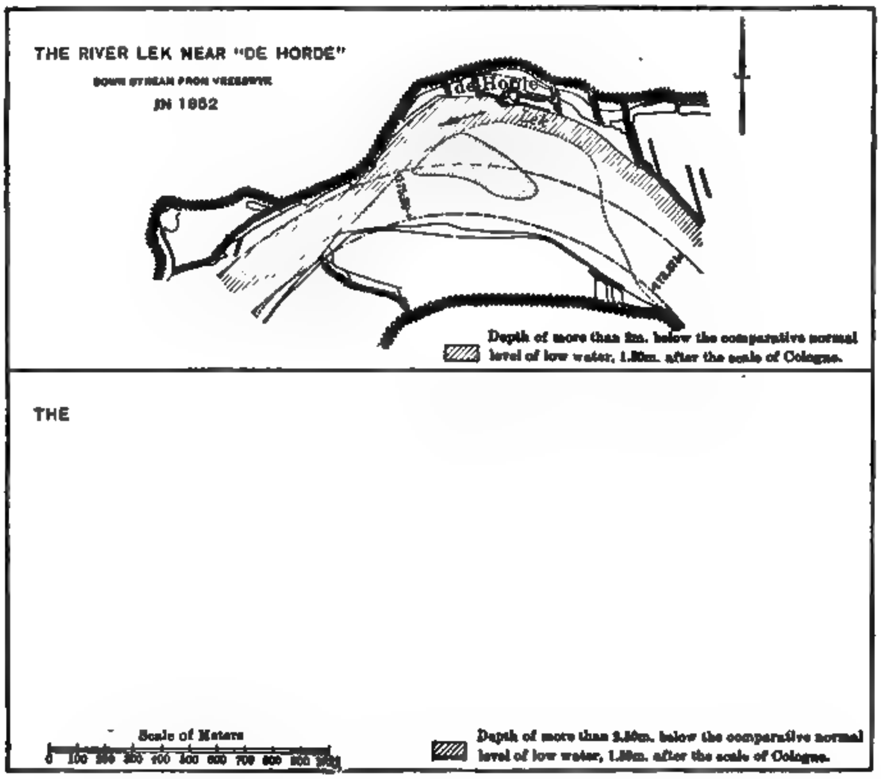


FIG. 10.

a series of pools separated by shoals, could be changed so as to secure a practically uniform slope by distributing through the pools the fall naturally concentrated at the shoals. This would in theory distribute the uneven depths also, making the pools shallower and the shoals deeper, and thus benefit navigation. As experience was gained, however, the error of these beliefs gradually became apparent, and it came to be recognized that flowing water moves under the guidance of natural laws which produce in their combinations complex results which cannot be interfered with lightly.*

*For a summary of the results of the early experiences see "The Improvement of Non-tidal Rivers," by Col. Wm. E. Merrill, Corps of Engineers, U. S. A. For references to later improvements see "Rivières à Courant Libre," by F. B. DeMas, Reports of the German and United States Governments, etc.

The regulation of a river for navigation may be divided under two broad headings: the correction of alignment, and the modification of depth. The former can be illustrated best by taking some examples of irregular channels of the types usually met with. Fig. 11 represents a bend obstructed at *B* by a point of hard material, the main axes of flow being shown by the arrows. The tendency of the river, however, is as represented in Fig. 12; the water endeavors to pass along the bank in a smooth, uninterrupted curve, and the object of the improvement should be to approach this condition as far as practicable. If a moderate increase of depth alone were desired, and a sudden turn in the channel would not prove objectionable for boats, it might be secured by cutting off the low-water flow of the left-hand channel by means of a dike, as at *BB*, Fig. 11, assisted if necessary by dredging. If, however, the improvement was to develop to the full the possibilities of the river, a more radical treatment would

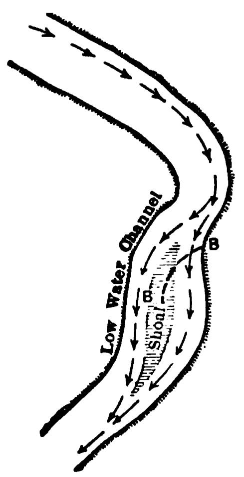

FIG. 11.

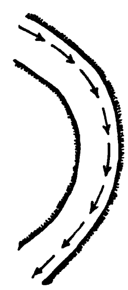
FIG. 12.

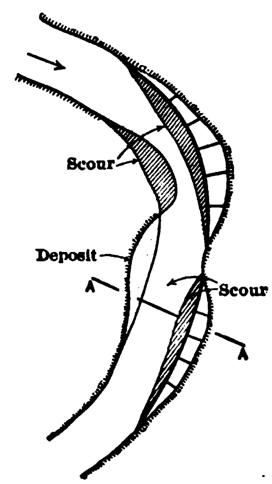

FIG. 13.

be required. Figs. 13 and 15 show the two methods, one of which would usually be adopted, the former consisting of the building of training walls with cross dikes without removing the hard point, and the latter, of the removal of the obstruction. Fig. 13 flattens the curve and tends to a slight modification of depth, and Fig. 15 keeps it of the same radius as before. The method of Fig. 13 would be used if the natural curvature was too sharp for navigation, or if the land behind the dike was to be reclaimed, or if it was undesirable to cut into the point. It would result in a considerable scour along the foot of the upper training wall when it began to shut out the water, and erosion would also take place opposite, since the river would have to force its way through a contracted area. Similarly, but to a less extent, scour would occur along the lower wall and also on the shoal just below the point, owing to the diversion of current. However, as this scour began to facilitate the flow, the river would tend to concentrate in the deeper channel, and would begin to deposit along the bank as shown in Fig. 13. The resulting plan and cross-section would be about as shown in Figs.

12 and 14. The accompanying cuts of the River Lek (Fig. 10) in Holland show an example of a curve thus regulated and others will be found in the cuts of the regulation of the Rhone. (Pls. 1 and 2.)

If the method of Fig. 15 were adopted, and the point cut back to the line of the bank, the result would be that the river would make deposits opposite, as shown, until it had contracted and readjusted its channel to the least area needed to pass the flow. The final plan would again approach that of Fig. 12. In this case, however, unless the bank above and below the point was protected from further erosion, the conditions shown in Fig. 11 would tend gradually to re-establish themselves.

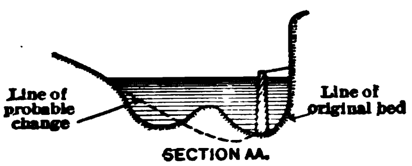


FIG. 14.

Care must be taken in planning such improvements that property be not unduly injured by the deposits. If wharves are in existence where the shoaling is expected to occur, the water may become too shallow to permit their use. Such a case once occurred on a tributary of the Hudson River. The conditions were somewhat similar to those shown in Fig. 15, the wharves being situated opposite the point and on the right-hand channel (Fig. 11). When the point was removed, the river began to

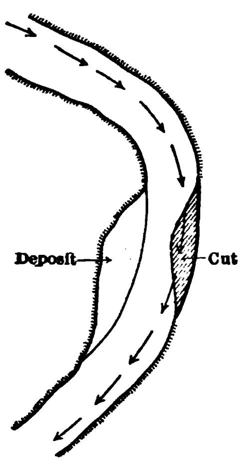


FIG. 15.

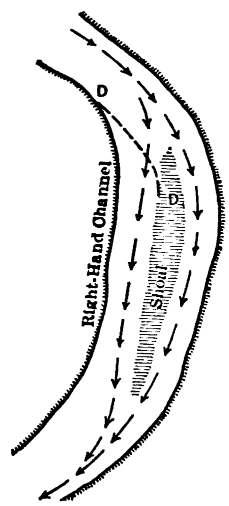


FIG. 16.

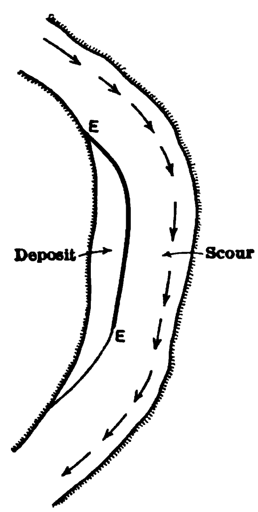


FIG. 16*a*.

establish its new channel limits and to assume the regular curve shown in Fig. 12, with the result that the depth in front of the wharves became seriously reduced. It may be noted that with these points the deepest channel is usually the one trending to the opposite shore, as in low water the current is deflected and scours in that direction.

Fig. 16 shows another combination often met with, where a bend widens with a corresponding bar in mid-stream, and the flow divides as before into two channels. The tendencies, as in the case of Fig. 11, are towards the even curve shown in Fig. 12. In such a case a training dike across the right-hand channel, as indicated at *DD*,

Fig. 16, assisted by some dredging of the bar, may correct the irregular flow sufficiently to allow the river to scour out the left-hand channel—the one it wishes to use—and thus establish its rational course. Should this prove insufficient, more pronounced and

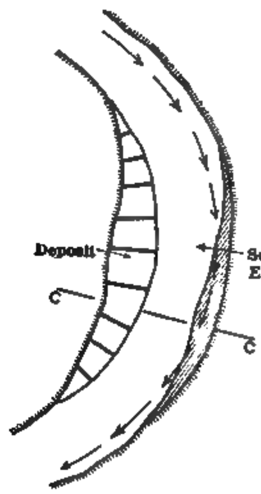


FIG. 17.

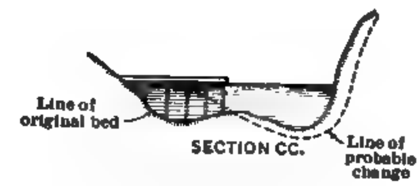


FIG. 18.

probably satisfactory results might be obtained by extending the dike down the shoal, as at *EE*, Fig. 16*a*, or the right-hand bank could be built out with spur dikes, as shown in Fig. 17. This would force the river over on the opposite bank and the shoal would tend to scour until an outline approximately resembling that of Figs. 12 and 18 had been obtained, by which time the spaces behind the longitudinal dike or between the spur dikes (provided these were of a design which did not cause too much eddying in the flow) would have become largely silted up and an even bank line would have been established. With shoals of hard material dredging should be resorted to to assist the scour.

Fig. 19 is a combination sometimes met with of a hard point at *E* and a reverse bend, somewhat similar to the conditions of Fig. 11, but complicated by the presence of a crossing. Fig. 21, p. 59, shows the natural tendencies of the current, the greater part of which in

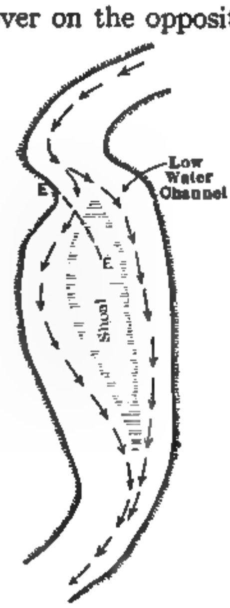


FIG. 19.

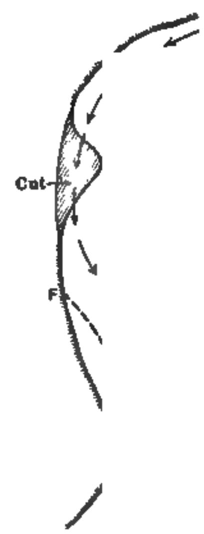


FIG. 20.

low water is deflected towards the bank opposite the point *E*, while in high water, when the influence of this point is less, it tends to follow the other bank until the crossing begins. A dike at *EE*, Fig. 19, would produce some improvement of

depth, as described for Fig. 11, but thorough improvement could be secured only by removing the cause of the evil and cutting back the point. If this were done completely, the river would immediately begin to scour out its proper channel, as shown in Fig. 20, and to fill out the opposite banks. With this type, however, the conditions are usually somewhat aggravated, and a dike, as *GG*, might be found necessary, as well as dredging, to give the flow a definite course in its preliminary work. If the case is an obstinate one, *GG* would probably have to be extended, say to *H*, and perhaps be supplemented by a second dike *FF*, so as to restrain within proper limits the low-water flow over the crossing and prevent it from spreading as it did before.

In Fig. 22 is shown a stretch complicated by a crossing so long that the water has partly lost its direction, while floods have built up islands in it which render the conditions worse. The natural line of flow would tend approximately to the direction shown in Fig. 21, and in such cases it will usually be found that the currents are gradually working their way across as shown in Fig. 22. Opinion seems to be divided as to whether the improvement of such reaches should begin at the lower or at the upper end. It is claimed by some that if the lower end is chosen and the lower channel opened first, the upper end will scour more easily; those holding the opposite opinion claim that if the improvement is begun at the upper end, the material loosened will be carried out of the reach instead of tending to settle in the lower portion as might be the case if the improvement were begun below. Fig. 23 supposes that a radical improvement has been decided on, and that the current is to be trained along the lines indicated in Fig. 21, and that the closing has been commenced at the upper end by building permeable closing dikes across the left channel and between the upper two islands. These dikes should cause only a slight obstruction, but one sufficient to cause the river to deposit sediment below them, thus commencing the permanent diversion of the flow. As this channel began to silt up, the river would commence to deepen the other one, and would probably widen it also by erosion of the right bank. The dikes would perhaps have to be increased in height after the changes had progressed to a safe point, and the processes would continue until the improvement had reached the final stage. Long before this was attained, however, the closing of the lower portion of the same channel would be begun, followed by that of the channel to the right of the last island, and the obstruction caused by the dikes should be increased gradually as the river adjusted itself to the new conditions. The evolution of these would probably have to be regulated by spur or training dikes in some places and by revetment in others. The results would be somewhat as indicated in Fig. 25, and the new channel would resemble in general outline the one shown in Fig. 21. If the river was of comparatively stable bed and carried little sediment the closing dikes could be made of a more solid character, but they should be built up by degrees in either case. Where the distance between the bends was considerable, it is probable that additional training works would be needed along the crossing, in order to prevent the water from spreading into a shallow channel.

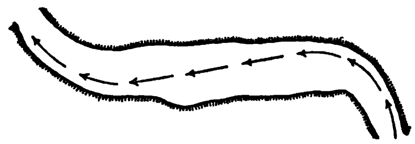

FIG. 21.

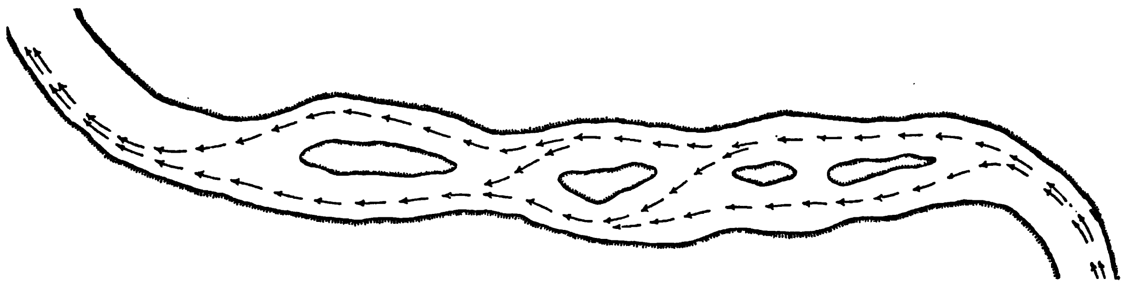

FIG. 22.

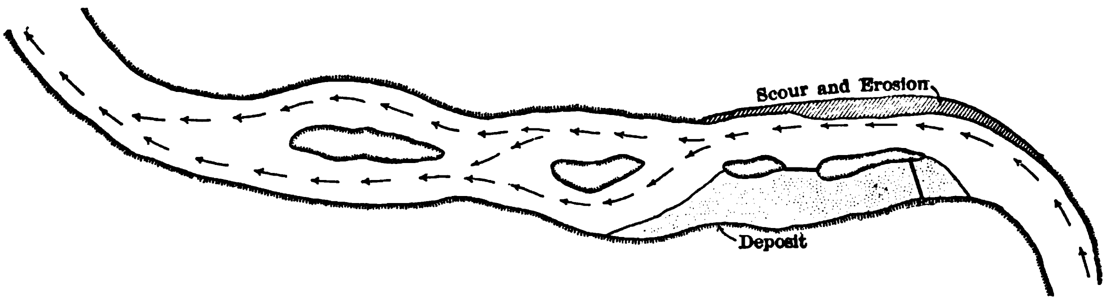


FIG. 23.

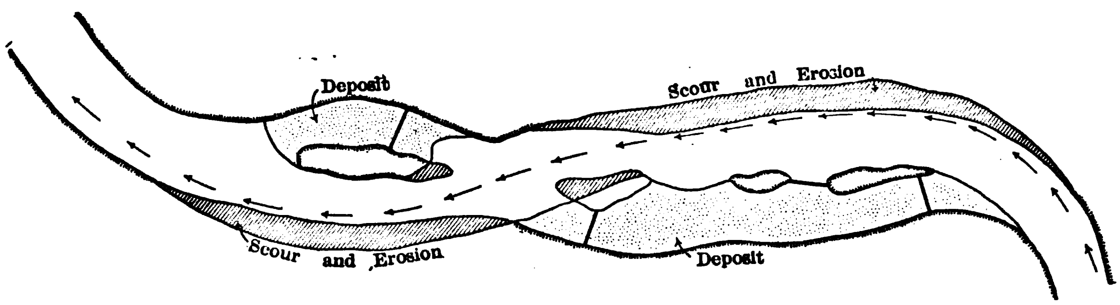


FIG. 24.

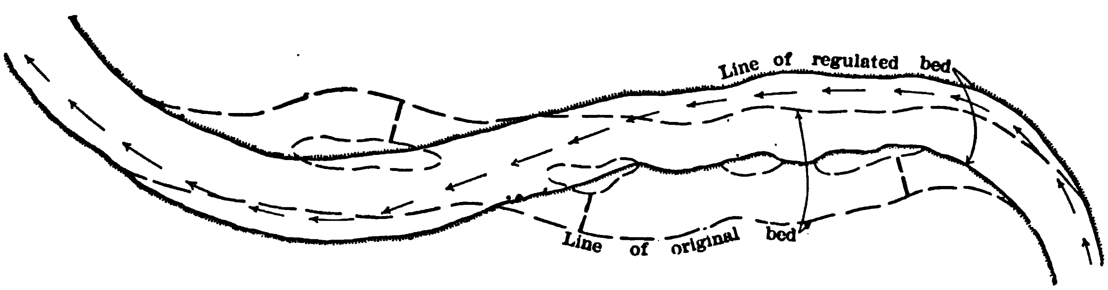


FIG. 25.

NOTE.—The probable changes in the bank lines are exaggerated in order to indicate more clearly the general tendencies.

The general plan should be to evolve as a whole the elements of the new channel, since the success of one part is dependent on that of the next, and if one change is allowed to progress too far, the currents may get beyond control and disorganize the whole work. In many cases it would be found that a thorough regulation as just described would be too expensive, or perhaps inadvisable, and that more economical or better results could be obtained by utilizing one or more of the secondary channels. It is of course impossible to follow successfully any hard-and-fast rule in such work, and each locality must be judged by its own possibilities, governed by the principle that the improvement should coincide as far as practicable with the natural tendencies of the stream.

The regulation of such a stretch should proceed slowly, as there would be a large amount of scouring to be done, and if the river were confined too suddenly its forces would tend to undermine and destroy the works, or would vigorously attack the banks in the attempt to regain free passage. As strata of sand are liable to exist in the banks at any point, the exposure of one of such seams to a strong current would lead to rapid caving, or if a hard point became exposed the resulting eddies and cross-currents might prove equally prejudicial to the improvement. Where dredging can be employed, however, the work of the river itself can be greatly reduced, and it will probably adjust its flow to the new channel without creating special difficulties. The material removed should be placed in the localities where deposits would tend to accrue naturally. This method of assisting the scour has come to be a necessity for channel improvement where speedy results have to be obtained. Where a considerable length of river has to be regulated, the amount of material to be displaced is often very great, and if the natural forces alone are relied upon, not only will their work be unduly prolonged, but the removal of so much sediment—which must be deposited, temporarily at least, further down—may also cause serious and unlooked-for changes in the river-bed below. Dredging will not only relieve the currents of any sudden addition to their labors, but it will assist from the outset in directing them in the new channel, and by placing the material removed on the banks or in backwaters, it will prevent the danger from new deposits below. In some of the improvements of the Rhine outlets the attempt was made to form the new channels by scour alone, but this was found slow and unsatisfactory, and the depths became irregular and uncertain, some places becoming too deep, while in others the scour was insufficient, and dredging was finally resorted to for the completion of the work.

The accompanying cuts of the River Waal (Fig. 27), one of the outlets of the Rhine, show a stretch improved in 1889 and similar in certain features to the one illustrated in Fig. 22. It is of interest to note how in the unimproved river the channel swung from side to side, and how it displayed the same tendency when confined between the artificial banks. The cuts of the Upper Merwede, another outlet (Fig. 26), show how a long bend, filled with shoals and short crossings, was transformed into a satisfactory

channel. This was carried out between 1899 and 1904, the action of the spur dikes being accelerated by dredging.*

Stretches of river are occasionally met with which display the conditions of Fig. 22 in a more aggravated form. These consist of very wide reaches filled with bars and islands, created sometimes by the inflow of a steeper tributary above carrying an excess

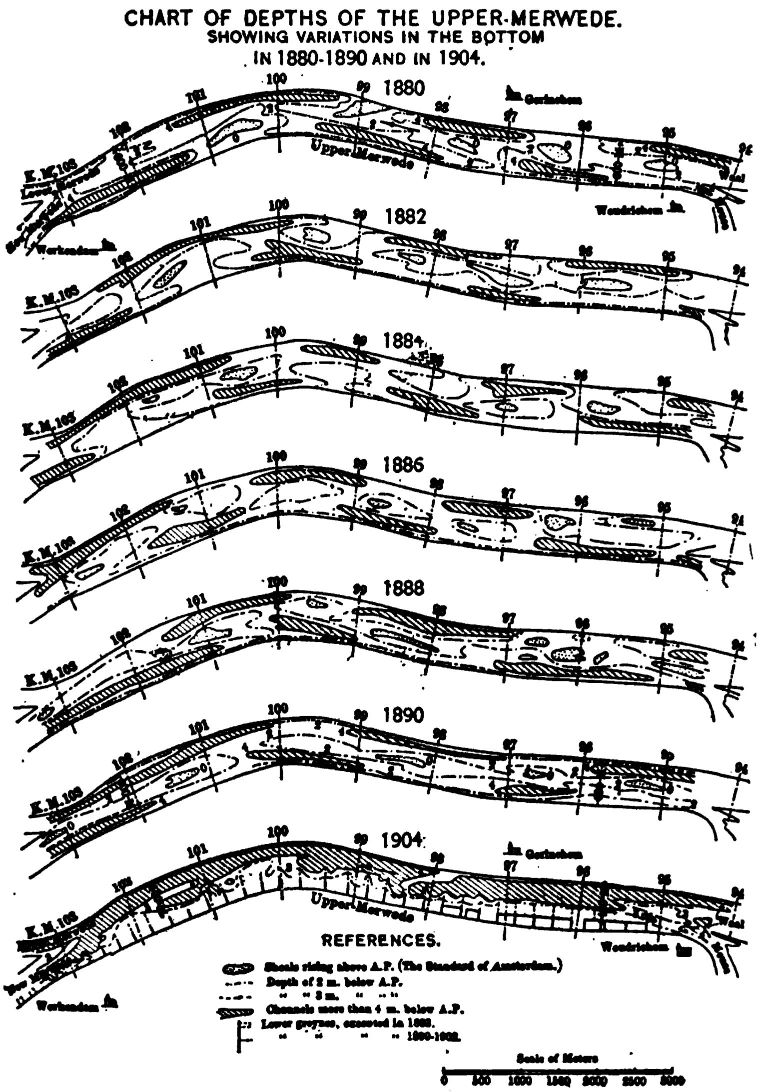


FIG. 26.

of sediment and sometimes by an unusual deposit of sediment by the main river itself, whose flood waters have lost their velocity in the wide reach, and have therefore been unable to carry on their burden. Such places are difficult and expensive to improve,

* An instructive article, "The Development of Natural Waterways in Holland," will be found in the Transactions Am. Soc. C.E., vol. liv, Part D (1905), describing the theories applied in regulating these rivers, and the results obtained.

as the low-water flow is spread out and apparently without definite direction. However, it will often be found to display certain tendencies in passing from the bend at the head to the bend at the outlet of the reach, not as a plain crossing, but as a series of minor bends and crossings as it gradually works its way across. By following these minor curves as far as practicable, and developing their channels to the best advantage, fairly good results will possibly be obtained. In extreme cases it may be necessary to create an entirely new bed, such as is shown on p. 81 in the cut of the regulation of the Danube near Vaika. These cut-offs, however, in rivers of steep slope are open to serious objections in that they shorten the natural alignment and therefore produce

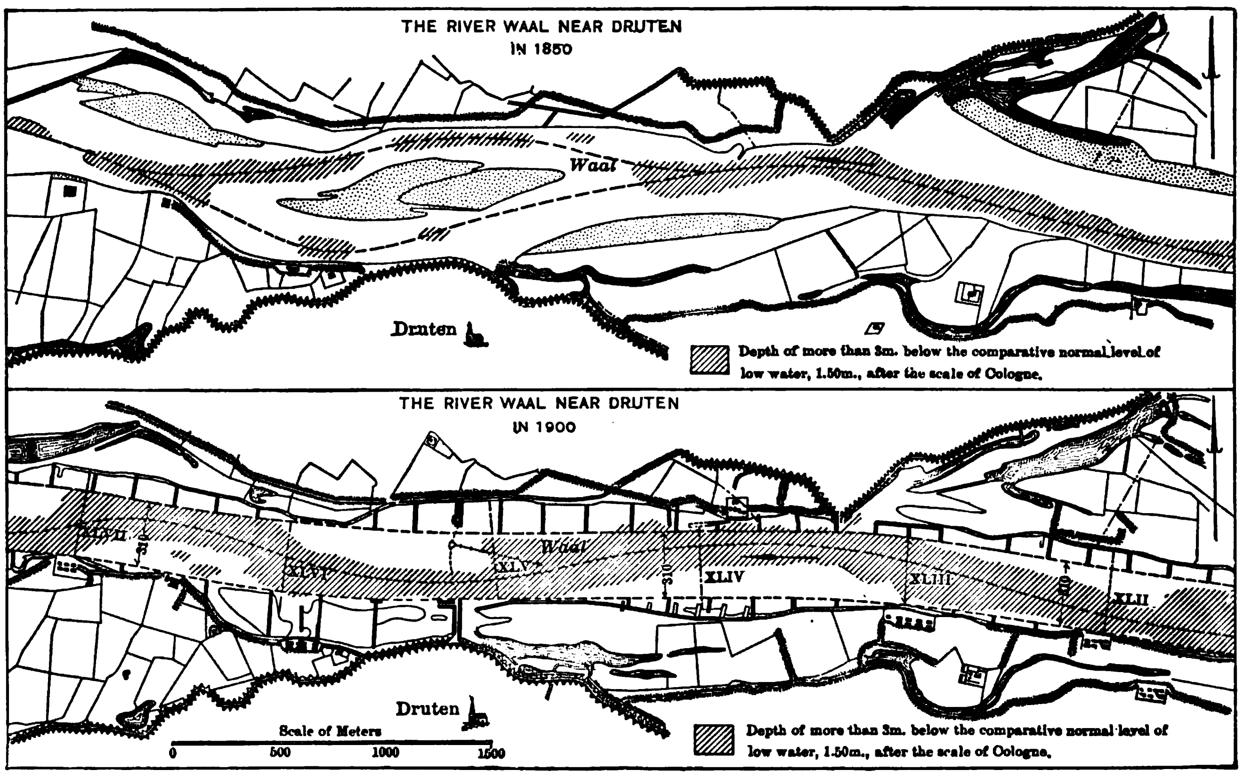


FIG. 27.

a steeper slope, with the probability of causing erosion with its accompanying changes. Their alignment and dimensions should be such as to cause a minimum of disturbance where the water enters and leaves the new channel, and unless the change is slight, the engineer must be prepared to use freely revetment and stonework. The accompanying Fig. 28 shows the improvement of a complicated stretch of the Merwede, one of the outlets of the Rhine, executed between 1850 and 1905, at a cost of $6,000,000.

Coming next to the second heading, the problem of the modification of depth, an example of a simple case will illustrate the general principles involved. Suppose a rock bar 600 feet wide, and with a fall of 2 feet per thousand to exist across a river from bank to bank, and that channel depths of 6 feet or more are found above

CONDITION IN 1850

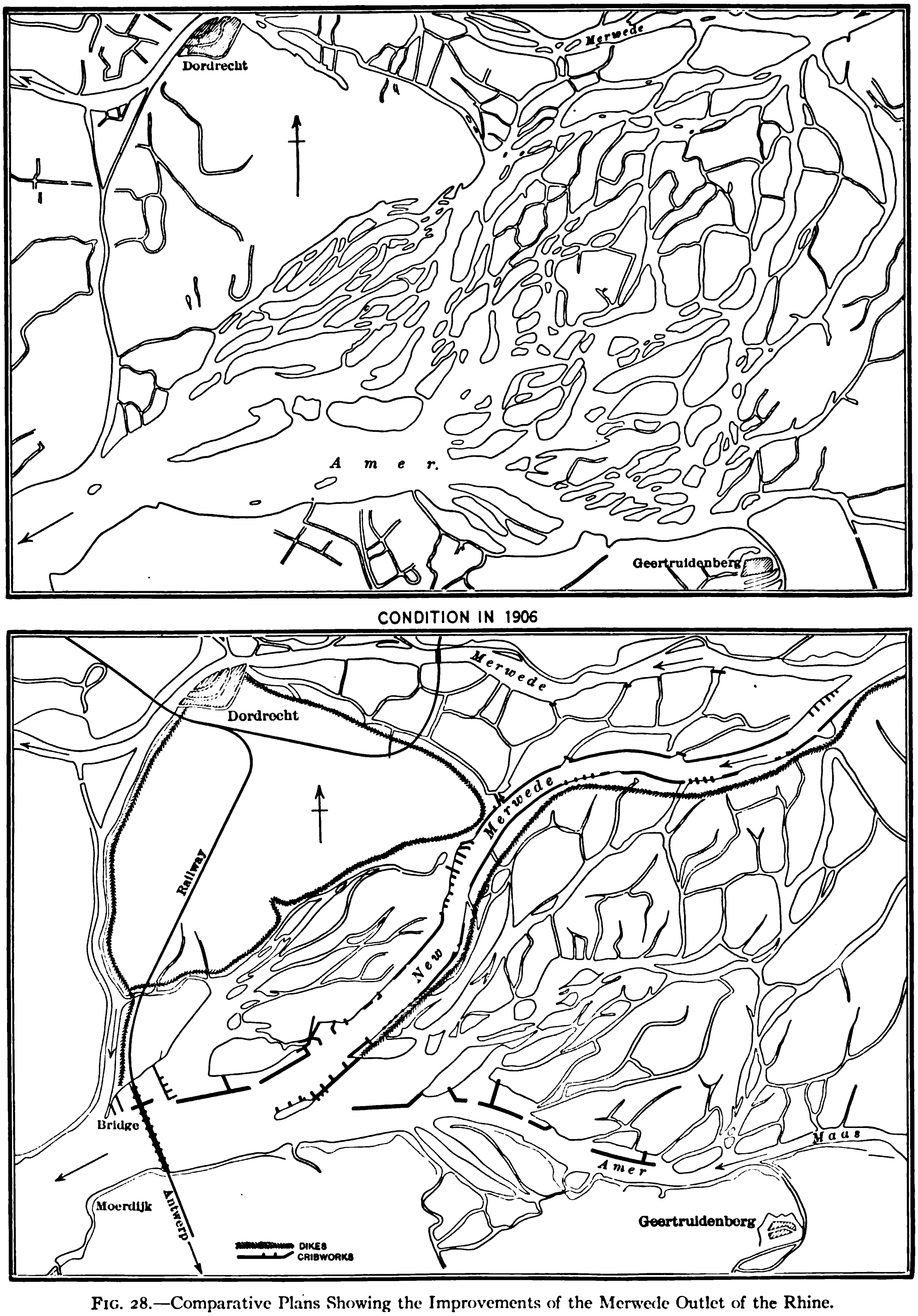


FIG. 28.—Comparative Plans Showing the Improvements of the Merwede Outlet of the Rhine.

and below, while a uniform depth of only 1 foot, giving a cross-section of 600 square feet, exists across the head of the bar. By excavating a channel 100 feet wide and 6 feet deep below the water surface, and by blocking up the remainder of the section of flow, an area will be obtained equivalent to the original one, but possessing a depth which will permit boats to pass with good draft what was formerly an insurmounable obstruction.

This is an example of the simplest case of such improvements, but while its elements are few, they involve several practical points to each of which due consideration must be given in proportioning the channel. Thus, while the area before and after improvement was the same, 600 square feet, the new channel, having a cross-section more favorable to the flow, will permit the water to run off more quickly. With its depth of 6 feet, therefore, it will carry off more water per second from the pool than did the old cross-section, and hence will lower the surface to an extent which may result in causing new obstructions. As this surface lowers, the depth in the artificial channel will of course fall also. Moreover, even should there be 6 feet of water at the entrance to the latter, the depth will decrease steadily down the cut, since the flow runs faster and faster, owing to the steep slope. This can often be seen exemplified on slope dams, where the depth on the crest is greatly reduced by the time it reaches the bottom of the slope. The resulting velocity again may be such that a boat will have difficulty in contending with the current. Lastly, it is of prime necessity to locate the new channel so that craft can enter and leave without risk, and without having to make any sharp turns either in transit or approach, especially if going downstream.

In this example we have assumed that the banks and the bed of the new channel were of rock, and therefore would not be changed by the altered flow of water. If they consisted of material liable to erosion, as sand or gravel, a new element would come in, and one of vital importance, as will be seen later.

**Limits of Improvement.**—According to what has preceded, therefore, the problems of improvement by regulation appear to comprise the following principal elements: (1) Supply of water; (2) Dimensions and curvature of channel; (3) Velocity of current; (4) Material of bed; (5) General location of channel.

(1) and (2) *Supply of Water; Dimensions and Curvature of Channel.*—The first and second elements are among the most important, since the boats must have a certain width and depth of channel, and unless the supply of water can fill it, navigation will be hampered. Before the general building of railways, when even the smallest rivers were in most cases the cheapest highways of traffic, depths of 18 inches and widths of 20 feet often proved advantageous where no greater could be obtained, and much work was done accordingly on streams now entirely neglected. The steady extension of railways and their advantages in handling freight have, however, changed conditions until navigation can compete only where traffic can be carried in large bulk, requiring an ample flow of water and width of channel. Thus regulation has been

gradually driven from the smaller streams, or, as with the Seine or the Ohio, has been or is to be replaced by canalization, and it is only on such rivers as the Volga, the Paranà, or the Rhine, that the methods can still be applied with commercial success. The dimensions of channel should of course be as great as the water supply will allow, and the ratio of depth to width must be determined by the character and size of boats. As a rule, where natural conditions will permit, a comparatively narrow and deep channel, if straight or of only moderate curvature, is preferable for traffic to a wider and shallower one, since with care a boat can keep its course in the former and can use special assistance at the most difficult places while ascending, and the risk entailed is more than compensated by the gain in draft or carrying capacity. If the channel has to be curved, it will require a greater width than where straight, since descending craft will be largely under the influence of the current and will tend to be carried against the dike, as well as to move crosswise in rounding the curve, and if an ascending boat is met at that point ample room will be needed for safe passing. The effect of wind is also often considerable, requiring additional care in handling.

The low-water channel dimensions for the Mississippi River below Cairo, adopted in 1896 and still in force in 1912, are 250 feet by 9 feet deep; those adopted for the Upper Mississippi vary from 300 feet by 6 feet near St. Paul to 1400 feet by 6 feet near the mouth of the Missouri. On the lower Volga in 1904, the channels dredged through the bars were made about 250 feet wide, with a least depth of 5.9 feet. On the Rhine the dimensions at mean low water vary from 1115 feet by about 10 feet at the Holland frontier to 754 feet by 6½ feet at Bingen. (See p. 79.) On the Rhone they vary from 390 feet at Lyons to 1300 feet near the mouth, the adopted low-water depth being 5¼ feet between Lyons and the maritime portion of the river, and 6½ feet in the latter. For the cataracts of the Danube minimum dimensions of 197 feet by 6½ feet were used, except through the rapids of the Iron Gates, where the channel was made 262 feet by 9 feet. The channel through the cataracts of the Dnieper was made 105 feet by 5 feet. On the Seine below Paris, canalized by movable dams for a channel depth of 10½ feet, the width of channel was made 165 feet in straight portions and about 200 feet in curves. With sharp curves, however, the latter width was found too small, especially for long tows going downstream. On the river divisions of the Barge Canal of New York State (1906 and after) the channel widths vary from 150 to 200 feet for the straight portions, without any widening except for curves of 5 or 6 degrees (1146 feet to 955 feet radius respectively). This canal was constructed with a depth of 12 feet. A less width than 100 feet is rarely advisable.*

A rule for curve-widening on canals was adopted at the International Congress of Navigation held at Vienna in 1886 as follows: "The amount of widening of the canal

* The Swedish Royal Commission on Navigation advised that for rivers the smallest wet section of the channel should be about six times the wet section of the largest vessel, and for slackwater canals in rock cuts about three times, and in earth from 4 to 4½ times the corresponding section. The least depth recommended should exceed the draft of the largest vessel by 20 per cent in rivers, and by 10 to 20 per cent in canals. (Int. Cong. Nav., 1912.)

bottom shall be equal to twice the versed sine of the arc whose chord is equal to the length of the longest boat." Thus for a curve of 1300 feet radius and a boat 220 feet in length, the widening would be about 10 feet. For navigation on a river of much current and sharp curves this proportion should be at least doubled. The Belgian engineers use the formula $W=\frac{a^2+b^2}{R}$, where $W$ is the widening required, $a$ and $b$ are the half-lengths of the two boats meeting at the curve, and $R$ is the radius of the curve, all dimensions being taken in feet or meters.*

Another rule, deduced from observations of steamships rounding curves in New York Harbor and in the St. Mary's River, Michigan, is $S=\sqrt{R^2+L^2}-R$, where $S$ is the increased space actually occupied by the boat (no allowance for clearance being included), $R$ is the radius of the curve, and $L$ is the length of the boat, all dimensions being in feet.

The curvature to be adopted must depend more or less upon the indications afforded by the river itself, and the natural radii will be found to vary with the size of the stream and in different parts of the stream. Thus the adoption of a radius which on a large river would permit the maintenance of a good and stable channel, on a small one would result in a succession of sand bars and shallow depths. On the Mohawk River division of the Barge Canal just alluded to, where many miles of dredging had to be done through the bed of sand and gravel, the sharpest natural curve was found to be of about 1433 feet radius (4 degrees). Where practicable the dredged channel in such curves was eased off with curves of 3, 2 and 1 degrees until it coincided with the adjacent tangents. On the regulation of the Elbe the radii adopted for the upper portion of the river were from 1000 to 2600 feet, and on the lower portion from 1590 to 5200 feet. On the Vistula they were from 2600 to 6600 feet for the upper portion and from 5200 to 8500 feet for the lower. The radii of the sharpest natural curves on the Ohio which can be passed with careful handling by descending coal fleets vary from about 1500 feet near Moundsville, 101½ miles below Pittsburgh, to about 2500 feet at a point 671 miles below the same city, the lengths being measured to the middle of the river. The radius of the outer limit of the channel would be from 200 to 500 feet more. The Mississippi between Cairo and New Orleans has minimum curves with radii from 4000 to 4600 feet. Along these, however, the erosion is excessive, while there are other curves with radii of 6000 to 7000 feet along which the river flows with much less disturbance. It is stated that where the radius exceeds 10,500 feet the erosion becomes slight. At New Orleans is a curve with a least radius to the center of the channel of about 4000 feet and a maximum depth of 204 feet at low water, and between there and the sea the sharpest curve has a radius of about 4600 feet and a maximum depth of

* International Congress of Navigation, 1912. Paper by C. Valentini. This writer states that experience has shown the desirability of a least radius of about 1650 feet for boats of 400 to 600 tons traveling on rivers. On canals where there is practically no current this radius can be reduced to about 660 feet. For boats of 300 tons these limits can be reduced one-half.

130 feet. Ocean steamships of great size pass this curve without trouble, as the channel is some 2000 feet in width. Where the channel is to be through rock and is of considerable length, the upper end is often widened so as to ensure a full supply of water throughout. Unless this is done, the increasing velocity will reduce the depth as previously described. Certain authorities state that to secure a constant depth and velocity in such a channel under ordinary conditions of slope, etc., it should decrease uniformly in width about 1 foot per 90 feet of length.

NOTE.—A table of the discharges, etc., of various rivers will be found at the end of Chapter I.

(3) *Velocity of Current.*—The velocity of the water in the new channel, apart from its effect upon the bed, as described in following paragraphs, should be as moderate as practicable, so that boats may ascend easily. At the cataracts of the Danube the artificial slope varies from 5 feet to 13 feet per mile, resulting in velocities of 6 to 10 feet and more per second, the channel depths being from 6½ to 9 feet. At the cataracts of the Dnieper the slope was limited to 10½ feet per mile, the mean being about 5¼ feet. The depth of water was 5 feet, the limit of velocity being about 6 feet per second. On the Rhone the maximum velocity in improved channels varies from 3.3 feet to 7.4 feet per second in low water, and in high water from 7.4 feet to 13.1 feet. A maximum low-water slope of 10 feet per mile was employed. As a velocity of 7½ feet per second means a speed of about 5 miles per hour, it will be seen that a loaded boat must have ample power in order to move against it. On the Rhine, the Danube, the Rhone, the Elbe, and several other rivers in Europe, a cable or a chain is laid along the bottom of the channel in some of the swiftest places. To use it towboats of special construction are employed, provided with large sheaves over which the chain is passed. These sheaves when set in motion wind in the chain and let it drop back into the river, thus hauling the boat upstream. The method is also in use on parts of the Seine and of certain other canalized rivers, and is employed as one of the regular systems of slack-water towing. On the Elbe such a system extended in 1906 from Hamburg to Aussig, a distance of more than 370 miles. In America, swift places are sometimes overcome by a line being carried ahead and fastened on the bank, and the steamboat then hauls it in by her capstan and in this way assists her propelling engines. On certain streams in Europe where towing was done by horses, it was found that where the slope exceeded 1 in 2000, upstream navigation became very arduous. On the river Lys, in Belgium, where this slope existed, the difficulty was much lessened by the presence of aquatic plants which retarded the velocity of the current, and which it was strictly forbidden to cut.*

(4) *Material of Bed.*—It has been described in Chapter I how a river when falling commences to lessen the deposits it has made on its shoals by scouring them away, carrying the material from the upper end and depositing it in the slower current below.

* "Civil Engineering," Law and Burnell.

The same action takes place when a shoal is regulated by dikes. The first effect of the contraction is a temporary raising of the water surface at the head of the shoal, proportionate to the change in the area of flow. This steepens the slope and increases the velocity, and if the change is sufficient to produce scour, as is the intention, the material begins to be washed away and to be deposited at the foot of the shoal, and the erosion ceases only when a reduction of the slope has produced equilibrium again between the material of the bed and the velocity of the current. The dimensions to be given the regulated channel should therefore be such as to produce scour about to the depth required and no more, since if the velocity is made too great and the bed is not held by submerged sills or similar means, the water will continue to scour until its forces become balanced once more at the natural point, with probably the former unsatisfactory depth and a lowering of the pool level above. Thus a channel velocity of 10 feet per second, allowable in rock, would speedily wash out a shoal of sand, and would result in reducing the low-water level above as far as the next shoal, while the final depth obtained over the shoal would be little if any greater than in the original condition. This reduction of low-water level would in turn increase the slope at the next shoal upstream and cause scouring to begin, and the causes and effects would gradually continue from shoal to shoal until checked or balanced by natural or artificial causes sufficient to stop the scour, as a rock bar, etc. Thus on the Rhone at Lyons, owing to the diking of local shoals below, the low-water surface fell $4\frac{1}{2}$ feet between 1858 and 1874, and on the Garonne, over an improved stretch about 30 miles in length, it fell an average of 4.3 feet in less than forty years, the bed scouring down at the upper end and filling up below.* On the Mississippi at St. Louis, the level was reduced from similar causes $2\frac{1}{2}$ feet in twenty-five years.

These results occur when a considerable length of river is subjected to improvement without sufficient allowance for the instability of its bed; the upper end is scoured down and the lower end fills up. An instance of it is given in Fig. 29, showing the results of attempts to improve the "Canal de Miribel."† This was a section of the Rhone above Lyons, about 11 miles long, with a mean slope of 4.7 feet per mile. The stream was divided into several shallow branches, and low-water navigation was in consequence very uncertain. The works of improvement, finished in 1857, concentrated the flow into one channel by closing the secondary arms and attempted by means of dikes to secure a low-water depth of 5 feet. At first navigation was greatly improved, but as the scouring progressed the summer water surface at the upper end gradually fell until the foundations of the dikes came into view, while the lower end filled up until the water invaded the tow-path. The mean slope was reduced to 3.7 feet per mile, but it was very irregular, doubtless owing to local aggregations of coarser materials, and the extremes varied from 0.6 foot to 7 feet per mile. The minimum low-water depth had

* See p. 65 and after of "The Improvement of Non-tidal Rivers," Col. Wm. E. Merrill, U. S. A.
† From "Rivières à Courant Libre."

increased from 2.7 feet to 4 feet, and the mean depth had become 5.7 feet, but it was too irregularly distributed to be properly available. Above and below the improved stretch of river serious changes were caused in the natural conditions. The slope of the six miles above increased from 4 feet to 5 feet per mile, and of the two miles below, from 2.6 feet to $5\frac{1}{2}$ feet per mile. The height of the dikes was then reduced, which sensibly lessened the progress of the scour, but it was reported in 1900 that equilibrium of the bed had not yet been re-established.

The effects of change of slope are discussed further in Chapter IV, under the heading "Amount of Contraction."

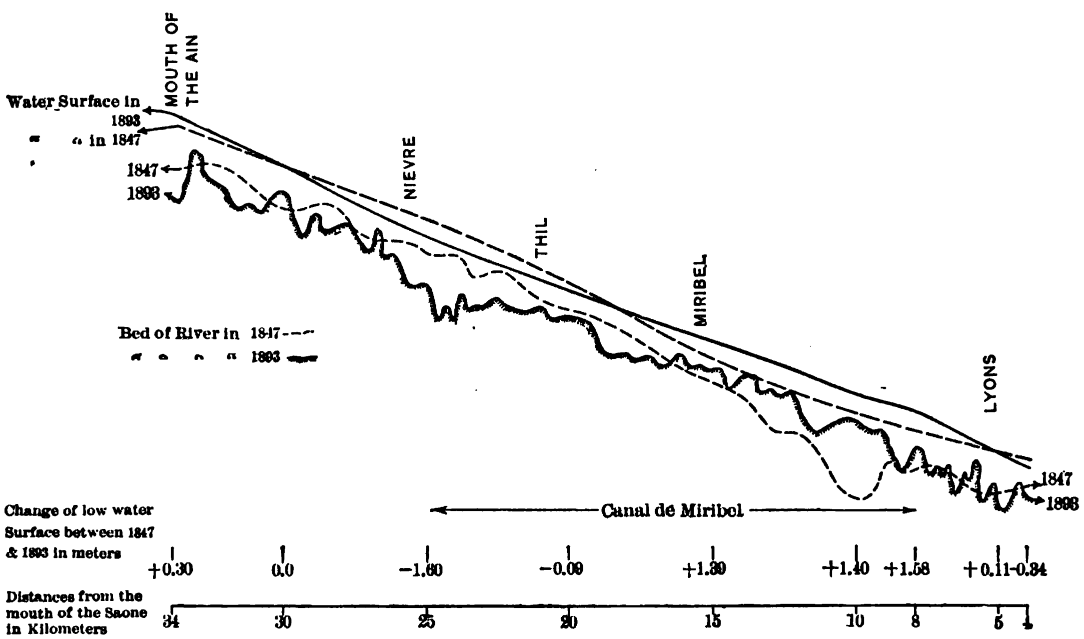


FIG. 29.—Profile of the Rhone above Lyons, Showing the Changes in the Portion Known as the "Canal de Miribel," between 1847 and 1893.

(5) *General Location of the new Channel.*—In the location of the new channel two aims have to be fulfilled: the one, to render the passage of boats secure and, if possible, easy; the other, to avoid undue change in natural or artificial conditions. With large rivers the curvature is usually such that the first aim is accomplished in fulfilling the second. With small rivers the ends of the channel should be placed so that boats can enter or leave on a straight course or on an easy curve, and in line with the natural flow of the current; all eddies or cross-currents should be avoided. The amount of curvature permissible has been discussed on p. 65. The need for retaining as far as possible the natural locations is explained elsewhere in this chapter, and the final plan can only be decided after careful study, and should be carried into effect slowly and with the necessary modifications, so as to avoid the complications liable to result from undue haste to complete the works, or from the lack of a sufficient understanding of the conditions to be met.

The artificial conditions just referred to comprise such matters as the locations of wharves, buildings, or other property which might suffer by any change in the position of the channel. Due attention must be paid to securing proper approaches to wharves, etc., under the new conditions.

**Surveys.**—In designing any improvement by regulation, whether local or extended, it is of course necessary to have accurate surveys of the bank lines and the bed, and to obtain the discharge of the river at various stages near low water. The discharge measurements will show approximately the volume of flow to be depended on for navigation during seasons of ordinary low water, although every few years there will come very dry seasons when the flow may be insufficient for navigation of full draft. In making the surveys the first step is usually the ascertaining of the low-water elevations along the river. This must of course be done during the dry season, bench marks being set to which the elevations can at any time be referred. The surveys, which are referred to a fixed plane, as sea-level, can then be carried on later irrespective of the stages of the river. The maps for the studies for the improvement usually show the contours below water plotted in reference to the low-water plane at the points in question, so as to indicate the actual low-water depths. The contours under this method are thus not shown as level, but parallel to the slope of the water. For rivers of ordinary size a convenient scale for the maps is 1 to 2400, with contours 2 feet apart; large rivers will require a smaller scale, and one of such a size that the eye can grasp the general relative features of each locality. A light hatching of the contour of the desired depth will then assist in making plain what portions of the bed are too shallow, and where the needed depth already exists.

**Natural Indications for Improvement.** A careful study of the maps just referred to will throw much light upon the general régime of the river. There will appear great variations of channel—long and shallow curves, sharp and deep ones, irregular depths on crossings, and all the many effects caused by the differences between the elements of flow and bed. At many places, however, there will be found conditions of low-water channel closely approximating the ones desired, and furnishing indications as to the limits of depth, curvature and width to which the river can be trained. The study of these limits is of the highest importance, since they represent the solution by Nature of the problem of equilibrium between the various elements, an equation whose solution attempted without such help would be of great uncertainty. The more closely these limits can be copied or reproduced without too much change of the conditions existing naturally at the point to be improved, the greater will be the probability of obtaining satisfactory results. This will be found described at greater length in Chapter IV, under the heading "Amount of Contraction."

**Regulation of Rivers with Sandy Beds.**—There is one class of rivers whose improvement by regulation is often unsatisfactory: namely, those whose beds consist largely or wholly of sand. One of the chief reasons for this is that for the successful treatment

of a stream the bed must consist naturally of a series of shoals succeeded by pools of some depth. In floods the shoals build up, and the pools scour out; as the water falls, the shoals begin to scour, and unless there are pools below them deep enough to receive and hold the eroded material until the next flood comes to carry it off, the result will be that the scouring only makes the shoal longer by building out its foot, quickly reducing the slope and finally suspending all erosion. Dredging is rarely of use in such cases except for temporary relief, since every rise tends to fill up the cuts and to reproduce the same conditions as before. On the Upper Mississippi River, for a distance of nearly 70 miles below the mouth of a sand-bearing tributary, the Chippewa River, is found a succession of sand-bars with very little deep water between, and the river, though contracted by dikes, was unable by its own forces to produce a satisfactory low-water channel there, although above and below, under more favorable conditions, good results had been obtained with a similar treatment.

**Effect of Regulation upon Floods.**—This question was discussed at the International Congress of Navigation of 1900, and the consensus of opinion was that flood heights had been reduced upon such rivers as had been fully regulated. The matter is naturally difficult of absolute proof, as it would be impossible to state to what height a particular flood might have reached under the original conditions, but the evidence adduced from records of the Rhine, the Severn, etc., appeared to indicate a probable reduction of flood heights of from 1 to 3 feet.

**Reservoirs and Regulation.** See last paragraph of Chapter VII (Storage Reservoirs).

**Formulas.**—The formula generally employed for determining the dimensions of an artificial channel, and which was used for part of the Rhone, for the regulation of the Meuse prior to its canalization, and elsewhere, is the Chézy equation:

$$v = C\sqrt{RS} \quad \text{or} \quad R = \frac{1}{C^2} \cdot \frac{v^2}{S}, \qquad (1)$$

and the formula for discharge, $Q = Av$, where $R$ = the hydraulic mean radius, $v$ = mean velocity in feet per second, $S$ = fall or slope per foot (expressed in feet), $A$ = area of cross-section in square feet, $Q$ = low-water discharge in cu.ft. per second, and $C$ = a coefficient depending on the roughness of bed, etc.* (See also Chapter IV, "Amount of Contraction.")

The maximum velocity in an improved channel is usually deduced from the mean velocity $v$ by Bazin's formula,

$$\text{Maximum velocity} = v \times \left(1 + \frac{25.4}{C}\right). \qquad (2)$$

* Tables for values of $C$ can be found in Ganguillet and Kutter's work, in Trautwine's "Pocket-Book," etc. Convenient tables for finding the value of the factor $n$ which enters into the solution of this formula have been worked out by Mr. Irving P. Church (Wiley & Sons, publishers). $Q$ is usually taken as the ordinary low-water flow, or the low-water flow which may be expected in all but exceptionally dry years.

In the improvement of the Rhone under the project of 1878 it was proposed to secure a low-water channel having a minimum depth of $5\frac{1}{4}$ ft. over a width of 130 to 165 ft., the total width of the new channel being considerably greater. The general outline of the bed was assumed as a parabolic section of the second degree, where $H$ the middle depth (see Fig. 30) was equal to $\frac{9}{8}d$, $d$ being the minimum low-water depth desired. The ratio between the full width $L$ of the new or artificial channel and the width $w$ of the new low-water channel was taken as 3 to 1, which by integration gave the average depth (for $L$) $D=\frac{2}{3}H=\frac{3}{4}d$*.

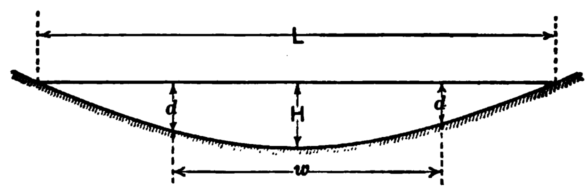


FIG. 30.

To find the width $L$ the formula first used was:

$$Di=bu^2, \qquad (3)$$

where $i$ is the slope per mile in inches and $u$ the velocity in feet per second, and $b$ is Bazin's coefficient for gravel beds, equivalent to $5.409\left(1+\frac{4.1}{D}\right)$.

This was then combined with the formula:

$$L=\sqrt{\frac{bQ^2}{D^3i}}. \qquad (4)$$

$Q$ being the discharge in cubic feet per second.

It is stated that on the Meuse the dimensions of channel actually realized closely approximated the calculated ones, until the absence of submerged sills permitted the scour to continue beyond the amount desired.

A study of the methods adopted and the results obtained in actual examples may prove of interest, and brief descriptions will be found in the following paragraphs of some of the most important works of regulation which have been carried out in different countries. A more complete description of certain features will be found in Chapters IV and V.

**Regulation of the Rhone.**—The early attempts for the improvement of this river were of local character, and often carried out by the landowners, and it was not until 1860 that the first comprehensive plan was adopted. The estimated cost of the proposed works was $7,460,000, and the depth in low water was to be $6\frac{1}{2}$ feet for the maritime por-

* "Improvement of Non-tidal Rivers," Col. Wm. E. Merrill, Corps of Eng'rs., U. S. A., p. 60.

tion, and 5 feet from there to Lyons. The improvements carried out under this program attained only a certain measure of success, partly owing to lack of funds and partly because they did not allow sufficiently for the erosion of the bed when the river was contracted. The increased currents gradually cut down the shoals and carried the material downstream (in the manner shown by the profile of the Canal de Miribel, p. 69), thus restoring in course of time the equilibrium between the slope and material of bed, and largely nullifying the benefits of the improvement. In 1878 a revised program was adopted, increasing the depth below Lyons to 5¼ feet, and allotting for the purpose $15,000,000.

The discharge of the Rhone is very variable, although it receives a supply, more or less constant, from Lake Geneva. The low-water discharge of 1884 just above Lyons was 4600 second-feet, and below the Durance, 13,000 second-feet. The high water discharge of 1856 at the same points was 190,000 second-feet and 490,000 second-feet respectively.

The distance from Lake Geneva to the sea is 324.7 miles, and from Lyons to the sea about 206 miles, with a total fall in the latter distance of 520 feet, or about 2½ feet to the mile. This fall is, in places, very considerable, as the following table shows:*

| Section. | Length. | Fall per Mile. |
|---|---|---|
| From Lyons to the Isère | 65 miles | 2.6 feet. |
| From the Isère to the Ardèche | 54 " | 4.1 " |
| From the Ardèche to the Durance | 35 " | 2.7 " |
| From the Durance to Soujean | 17½ " | 1.4 " |
| From Soujean to the sea | 34 " | 0.12 " |

On a few stretches the local slope runs as high as 21 feet per mile. The surface velocities during periods in which navigation is possible are in low-water from 3¼ to 8¼ feet per second; in moderate stages, from 5 to 11½ feet; and in high water, from 8¼ to 13 feet.

The bed of the river consists generally of gravel easily moved by a current, with occasional bars of rock, and boulders at the mouths of torrential tributaries. The size of the gravel varies from shoal to shoal with the slope, and the banks and bed for the last 34 miles consist of fine sand, the coarser material having disappeared. This sand becomes finer as the mouth is approached. Over the greater portion of the river the gravel did not move under a mean velocity of 5 feet per second, while at those shoals composed of very coarse gravel much higher velocities failed to disturb the material.

Before improvement the depths often fell to 16 inches, while at the present time a least depth of 4¼ feet can be depended on, and during a period of 20 years an average was reached of 341 days per annum with a depth of 5¼ feet, or more. The relative gain in the period of actual navigation was estimated at 85 per cent, and in ease of navigation, 136 per cent.†

* "Rivières à Courant Libre," p. 356.

† "Rivières à Courant Libre," p. 358.

In planning the works directed by the law of 1860 the problems comprised the reduction of shoals and the improvement of difficult bends. The width of the artificial bed was determined partly by the slope and discharge, but principally by the rational method of observing the natural width and location at low water at the point to be improved, and by comparison with results obtained elsewhere. From the deductions made, standards of width, changed within small limits to suit the varying conditions, were adopted for long stretches of river. Between Lyons and the Isère these widths were from 600 to 650 feet; from the Isère to the Ardèche, 650 to 800 feet; from the Ardèche to Soujean, 800 to 1000 feet; from Soujean to the sea, 1000 to 1300 feet. The depth, as before mentioned, was to be 6½ feet on the maritime Rhone and 5 feet from there to Lyons. In laying out the concave dikes for the improvement of the bends, radii of less than 3000 feet were avoided where possible except for short arcs, while the maximum limit was 10,000 feet, as it was found that with curves of larger radii the channel tended to become uncertain. The tops of the dikes, which were usually composed of gravel covered with stone, and with a footing of fascines on the river side, were placed at 6½ feet above low water from Lyons to the Isère; at 8 feet, from the Isère to the Ardèche; at 10 feet from the Ardèche to Arles, and at 9 feet in the maritime Rhone. The results of these works, except as regards erosion of the bed, confirmed in general the theories of the engineers. They were, however, local and of limited effect because of lack of funds, and it became evident that to obtain any permanent benefit the entire river must be improved. This work was authorized in 1878.

The principles adopted in planning the new improvements were to preserve natural conditions as far as practicable, instead of attempting to force the river to flow in a channel of regular cross-section and regular slope as had been previously done. The curves were modified where necessary by easy lines, retaining as much as possible the natural locations and removing accidental obstructions, such as reefs of hard gravel or of boulders, while the slope of the bed was retained by building at the shoals submerged dikes or sills which the velocity of the water when increased by works of contraction could not scour down. Thus the artificial profile of the river consists of an irregular succession of pools and rapids as it did in its natural state, but the bed is now fixed. Gravel and sediment are carried down as before, but the works of contraction prevent shoaling, and although more or less deposit occurs on the submerged sills during the varying stages, a good depth of water is available at all seasons. The records of soundings, which are carefully kept at many localities, show that on some sills there is little change of the bottom; on others deposit takes place in floods and scouring as the water falls; on others deposit occurs on a falling river and scouring during a rise. The variations appear to be due to local conditions of contraction, etc. The new widths adopted for the channel were: between Lyons and the Isère, from 390 to 490 feet; from the Isère to the Ardèche, 490 to 660 feet; from the Ardèche to Soujean, 660 to 820 feet; and from Soujean to the sea, 1300 feet. These widths, as will be seen

by comparison with those given before, were with the exception of the last one much less than those adopted for the first project. The maximum height adopted for the dikes was 5 feet above the lowest water; this was later reduced to about $3\frac{1}{4}$ feet.

The surveys showed that the shoals could be divided into two general classes of good crossings and bad crossings, of the types found in all rivers, and exemplified in Figs. 31 and 32. The bad crossings usually were found where the river divided into two channels, or where a stable bar was found, or the banks were very irregular. The good crossings were accompanied by regular banks and easy curves, and by a long gradient over the shoal. They frequently carried a least depth of 5 or 6 feet. The problem with the shoals was thus to change the bad crossings into good ones, following the methods indicated by Nature. This was done by closing all secondary arms, so as to make a single channel; by training the river within artificial banks suitably located; and by preventing undue scour by the use of submerged sills.

In correcting irregularities of channel in the bends, the dikes were placed so that the water on leaving a crossing or point of inflexion would approach and follow the succeeding concave shore in a curve composed of arcs of circles, of radii gradually decreasing until the point of sharpest curvature (the so-called "summit" of the curve) was reached, when the radii began to increase until the water was finally directed towards the other shore over the succeeding crossing. In this improvement the natural methods and locations were closely followed, and the flow was trained, not by sinusoids or other complicated curves, but by a series of simple arcs of circles of varying radii (transition curves) which passed through the bends in a continuous and easy line of channel. The convex shore was built out by a series of spurs, or by spurs combined with longitudinal dikes (Figs. 32*a* and 33) where the river was too irregular or was too wide, so as to produce a uniform outline, and thus complete the side limits. The height of the longitudinal dikes was made such that they concentrated the water only as long as was necessary for securing the desired depth of channel; if made too high, the result was too much concentration as the river rose and a consequent danger of excessive scour. The maximum height found by experience to be desirable was about $3\frac{1}{4}$ feet above the lowest stage, the crest rising slowly from the upstream end of the dike until opposite the point of greatest curvature, where the maximum height was placed, from which point the crest began to descend until it reached the level of the river bed near the point of inflexion. These inclinations of crest were employed under the theory that the channel would have less tendency to follow the dike closely the lower the top was placed, and that in consequence the water would flow more easily in approaching or leaving the crossing. In order to prevent the undermining of the dike on the bank side as well as to cut off the flow of water behind it, and to stop eddies and currents over it dangerous to boats, a series of cross-dikes was built joining the main dike to the bank. (See Pls. 1 and 2.) They were placed pointing a little upstream, and sloping downwards towards the main dike, the slope being increased as the point

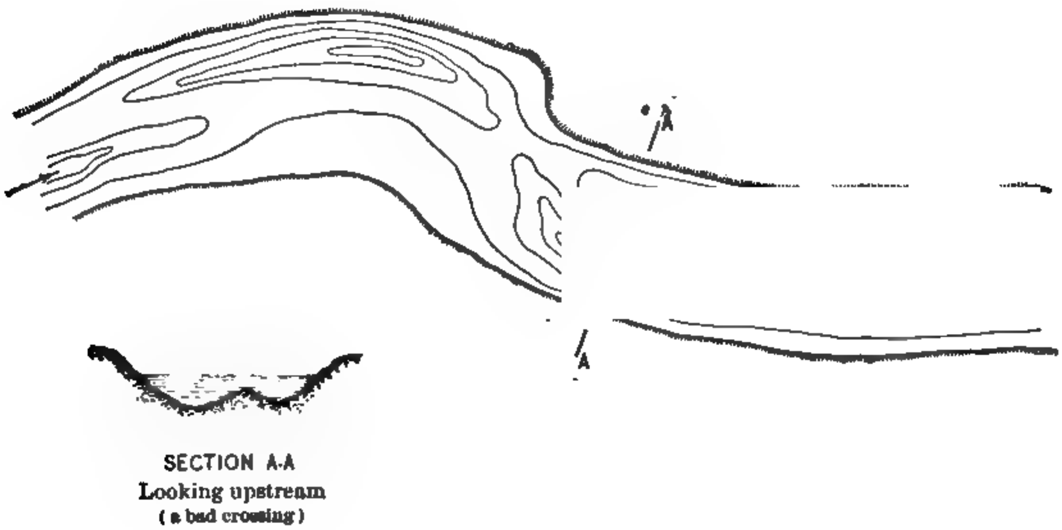


FIG. 31.

(FIGS. 31 and 32 are taken from the Survey Maps of the Mohawk River, New York State.)

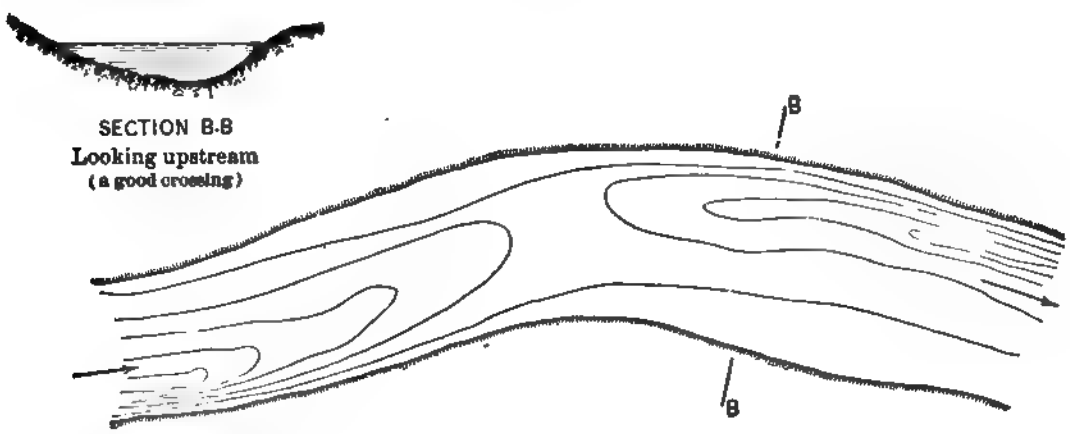


FIG. 32.


SECTION THROUGH A BEND
WITH SPUR DIKES

FIG. 32a.


SECTION THROUGH A BEND
WITH LONGITUDINAL & CROSS DIKES

FIG. 33.

of greatest curvature was reached, so as to throw the water back into the channel. In a few cases where the natural curvature and the slope of the river were slight, the longitudinal dike was omitted and the cross dikes alone were used.

In order to control the erosion at the crossing, submerged dikes or sills were placed in the bed of the river as required. Thus, in modifying a bad crossing (see Fig. 31, p. 76), the side depths near each bank had to be reduced so as to concentrate the flow near the center. The sills would thus start at or near one or both banks, depending on local conditions, such as the mobility of bed, etc., and would run on a downward slope from the shore out, gradually forcing the water away from the shore as it approached the point of inflexion, and bringing it towards the opposite shore further down, filling up part of the natural cross-section, and meeting the bed or dipping down below it where the new channel was to come. Their lowest point was placed on the axis of the latter. They thus formed a series of steps over which the water was to flow, rising gradually from the upstream one until the crest of the shoal was reached, when they began to descend, just as was the case with the bed of the river in a crossing naturally good, the conditions of which it was the object to reproduce.

Two of the most difficult portions improved, the passages of St. Alban and of Gerbay and Condrieu, are shown on Pls. 1 and 2.

The Rhone dikes were built almost entirely of riprap, those projecting into deep water being provided with aprons, as were also the submerged spurs. On the maritime portion permeable dikes of piles and wicker work were used in many places. Some of the works consisted of a gravel heart covered with rock, as on the Rhine. Where the submerged spurs joined longitudinal dikes, their side slopes were made comparatively steep, and were flattened out as they passed into the current so as to cause a minimum amount of reactions or eddies.

The works, as on the Rhine, were constructed gradually, so as to produce the desired modifications without forcing too sudden changes upon the river. By this method harmony was preserved between the effects of the various structures, and the scour was kept within control. The crests of the dikes were kept low until their effects could be ascertained, and the structures were then modified where necessary to meet the evolving conditions. Undue expense in rectifying the construction and the locations was thus avoided.

Traffic upon the Rhone has never been large in amount, partly because the river traverses districts which are chiefly agricultural, and partly because the rapid current renders upstream navigation costly and necessitates powerful steamboats. Projects have been considered at various times for canalizing the river and for building lateral canals, but up to the present it has not been believed that there would be a sufficient increase of commerce to justify the expense.

**Regulation in Germany.**—Experience in Germany, where regulation has been extensively applied, followed the same course of evolution as in France. The early

improvements were made with too little regard for the natural tendencies of the rivers, and it was only after long and costly efforts that the engineers realized the necessity of conforming to and retaining as far as possible the natural conditions. Professor Schlichting has tabulated the following principles upon which the German system of regularization has been largely based (see also p. 51):

1. Construction of spur-dikes on the convex banks, and of protection dikes and longitudinal dikes in the concave curves, with inclined submerged spurs to reduce unusual depths.

2. Transformation of slight sinuosities into straight lines, adopting the best possible location of the channel in the navigable bed.

3. Systematic alternation corresponding to the alternation of convexities and concavities, the spur-dikes being on one bank, and works of protection and dikes on the other.

4. Contraction of the width of the river by the advancing into the bed of convex curves, accompanied by a small flattening of concave curves.

5. Diminution of the mass of material transported in the navigable channel, to be accomplished by the consolidation of banks of alluvium in the convex parts.

It is stated that "by adopting a process of exploitation appropriate to the navigation, the Germans have succeeded in serving an enormous traffic with relatively small depths; the works of regulation have been in general relatively of small expense."

The minimum low-water depths adopted for some of the principal rivers were as follows: Vistula, 5.75 feet; Memel, 5.7 feet; Weser, 2.7 to 4.3 feet, according to location; Oder, 3.3 feet; Elbe, 3.0 feet. The works consist very largely of spur-dikes, which the German engineers appear to consider more suitable to their rivers under general conditions than longitudinal dikes. Thus on the Prussian Elbe there existed in 1888 some 6100 of the former, while the sum of the length of the latter was relatively small. Submerged sills are also extensively employed.

*The Rhine.*—The most notable example of regulation in Germany is that of the Rhine, which constitutes the largest and most important waterway in Germany. Rising in Switzerland, it drains all the Swiss lakes except Geneva and Joux, and receives in summer from the melting of the glaciers a copious supply of water for navigation. At Strassburg the low-water discharge amounts to 13,500 second-feet; at Cologne, to 20,000 second-feet; while at Emmerich, near the frontier of Holland, it is 26,800 second-feet. The maximum discharge is estimated as 353,000 second-feet. The total area of basin is 36,000 square miles, and the length from source to mouth is 722 miles, of which 434 miles lie in German territory, and 103 miles in Holland. From Basle, where the river enters Germany, to the sea the fall is 806 feet or 1.5 feet per mile, and varies from a maximum of 10.1 feet per mile to 0.63 ft. at the frontier. The slope, however, is irregularly divided. Approaching Bingen it is 0.83 ft. per mile, while below that city is a section through a rocky gorge where the fall reaches 2.51 feet per mile. (See

Fig. 1*a*, p. 10.) Further down near Coblenz it is 1.32 feet; between Coblenz and Bonn, from 1.47 to 0.91 ft.; from Bonn to Cologne, 1.23 feet; and from Cologne to the frontier, 0.98 to 0.63 ft. The maximum surface velocity at Basle is 13 feet per second and at Strassburg, 10 feet. The flood range at Cologne is about 29 feet.

Prior to 1850 the works on the river consisted principally of shore protection, but between 1850 and 1861 a system of regulation was carried out which resulted in creating a low-water depth of 3 feet between Cologne and Holland. This gradually became insufficient for commerce; traffic on the river could not compete with the railways, and it became evident that unless further improvements were made the boats would have to give up the struggle. Accordingly, in 1879 a law was passed authorizing a new project of regulation at an estimated cost of $5,500,000. The least dimensions of channel desired were as follows:

From Bingen to St. Goar: 295 to 390 feet wide and 6½ feet deep.

From St. Goar to Cologne: 490 feet wide and 8.2 feet deep.

From Cologne to Holland: 490 feet wide and 9.8 feet deep.

The width of channel just quoted was considered the least net width suitable to the accompanying depth; the width between the proposed bank lines was considerably more. In carrying out this project dredging was largely employed instead of relying on scour alone, as had been done with previous improvements, and it is stated that the desired depths were secured within the estimated cost, except at a few difficult points.

The river is divided into three general sections, the upper Rhine, 225 miles long, extending from Basle to Bingen; the middle Rhine, 99 miles long, from Bingen to Cologne; and the lower Rhine, 213 miles long, from Cologne to the sea, making a total of 537 miles. Navigation commences at Basle, but owing to the swift currents in this portion it does not become important until Strassburg is reached, 79 miles below. The upper Rhine was formerly divided at many points into several arms, and the natural width was considerable. It was improved by closing the secondary arms and by building parallel training walls from 660 to 820 feet apart, the lengths of these works being about 205 miles. At Mannheim the spur dike-system was begun, and the width between the proposed bank limits was increased to 980 feet. The middle Rhine was characterized by sandy beds from bank to bank, being in certain localities as wide as 3300 feet, and the channels were split up into secondary arms. In this section was the rocky gorge below Bingen, and between this point and St. Goar more than $1,375,000 was spent from 1880 to 1900 on blasting operations alone. The improvements along the middle Rhine were greatly complicated by the numerous towns and villages on each bank, whose access to the river had to be preserved irrespective of whether they were situated on the main or on secondary channels. Dredging had to be resorted to on a large scale to deepen such channels, and it was accompanied by spur-dikes and by revetments in order to prevent the erosion of the banks and the con-

sequent re-formation of bars. The lower Rhine was characterized by many sharp curves and by low shores and divided channels, and the improvement was carried out as in the other sections, by spur-dikes, revetments, and concentrations of flow.

It has been found that more or less dredging is generally necessary after each flood in order to maintain the required depths, for while the training dikes would eventually scour away the new deposits, their effects are slow, and dredging has to be used to supplement them. This is more especially the case in the sandy portions of the river, where the material is easily shifted. In those portions where the bed is rocky with alluvial banks, the channel depth is largely self-maintaining, except for the material washed in from the banks by the waves from the large steamboats. To prevent this long stretches of the banks have been riprapped. Where the bed is of fairly stable gravel, only moved by very high floods, some erosion occurs in the bends and deposits on the crossings, necessitating a corresponding dredging.

The dredging is done by contract, and as there is a great demand for the gravel, the contractors pay the State about one cent per cubic yard for the privilege of removing it. In some portions of the river, as near the large cities, restrictions have to be put on the operations in order to prevent the contractors from unduly enlarging the river section by removing too much of the deposits.

The result of the improvements was that the total business of the Rhine ports rose from 8,473,500 tons in 1879 to 37,295,100 tons in 1899, an increase of 440 per cent in twenty years, while the area of overflowed lands was reduced by the construction of levees from 350 square miles to 67 square miles, resulting in much benefit to agriculture. It was estimated that the saving in freight rates alone during the year 1900 amounted to about $10,000,000, or nearly twice the total cost of the improvements. The average annual suspension of navigation amounts to 42 days, of which 8 days are due to floods, 17 days low water, and 17 days to ice.

The traffic is carried on chiefly in barges of 150 to 3000 tons burden, handled by steamboats of sufficient power to tow them against swift flood currents. The barges and steamboats of recent construction have steel hulls and are designed with fine lines, so as to reduce to a minimum the resistance during towing.

Drawings showing some of the effects of the dikes are given on Pls. 3, 4, and 5.

**Regulation of the Elbe.**—This will be described in Chapter IV.

**Regulation of the Danube in Austria.**—The Danube rises in the Black Forest, having a total length from source to mouth of about 1800 miles. Its low-water discharge at the head of the delta is given as about 70,000 second-feet, and the flood discharge at the same point as about 1,000,000 second-feet. The average fall from Vienna to the mouth, a distance of 1060 miles, is 0.48 foot per mile. The river is navigable as far up as Ulm. Prior to any improvement, much of the channel was shifting and uncertain, and broken in some places by rapids which could only be passed during floods, while large areas of land were subject to overflow. The works of regulation consisted

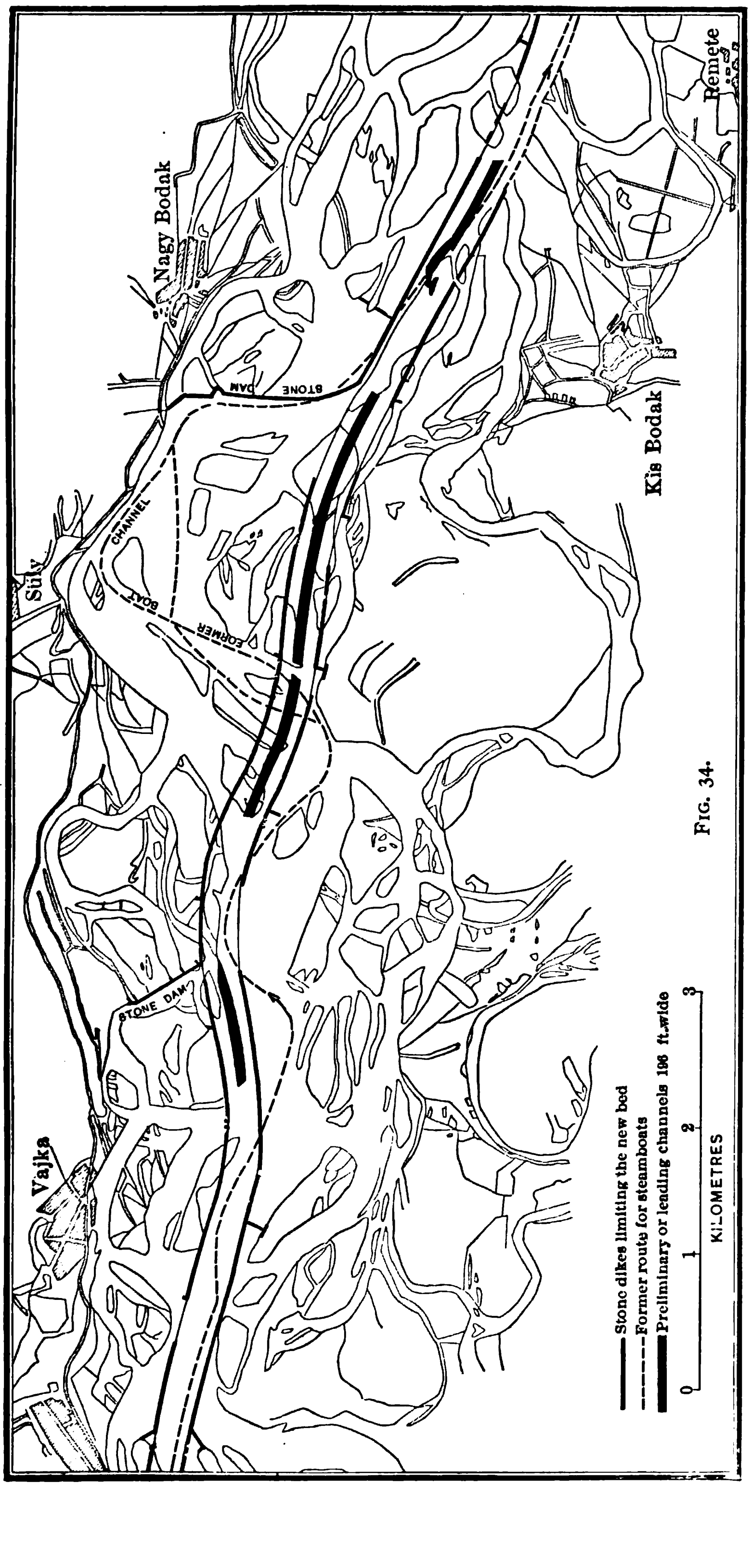


Fig. 34.

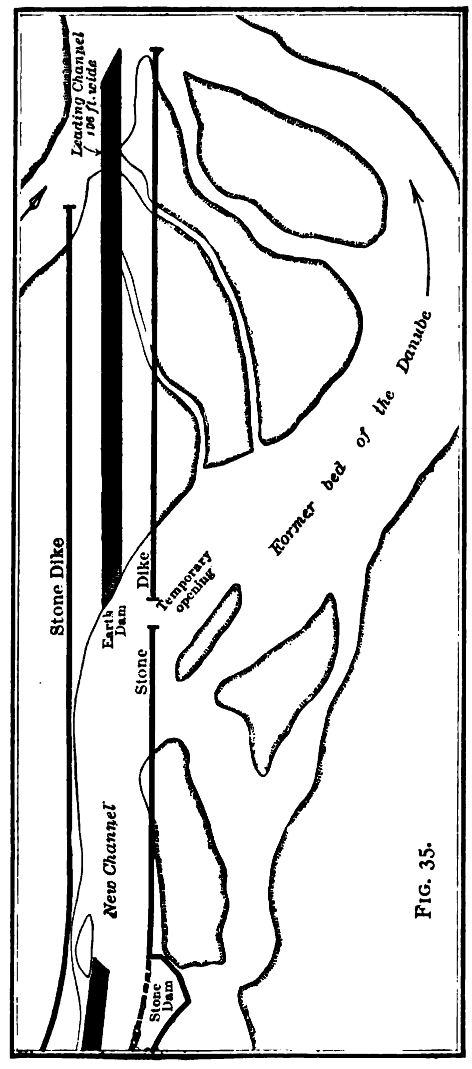


Fig. 35.

IMPROVEMENT OF THE DANUBE

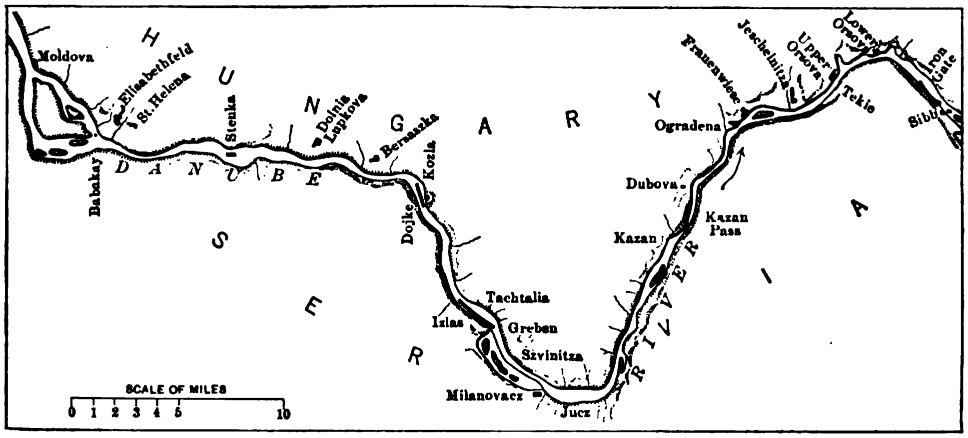


FIG. 36.—General Plan of the Danube from Moldova to the Iron Gates.

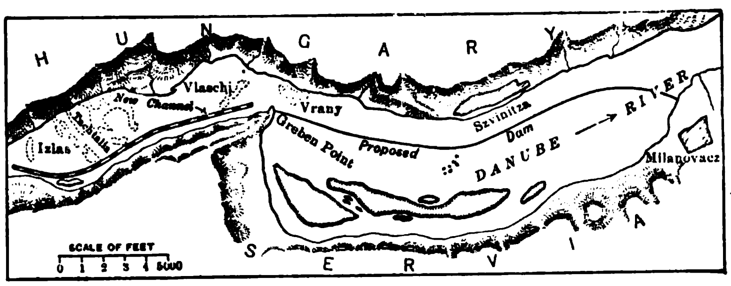


FIG. 37.

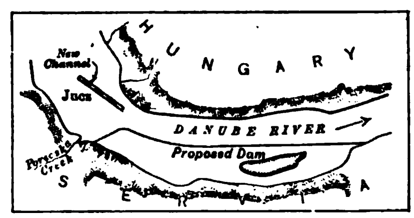


FIG. 38.

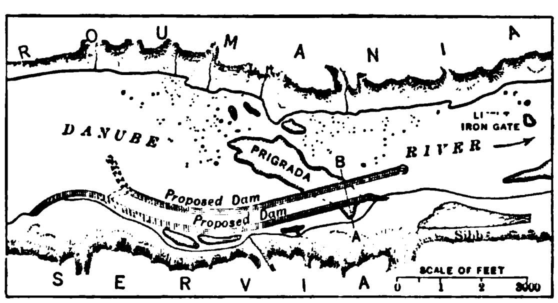


FIG. 39.

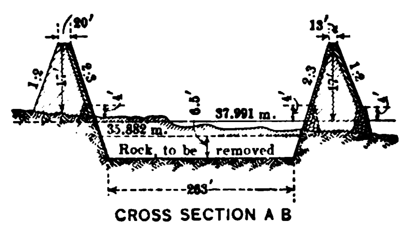


FIG. 40.

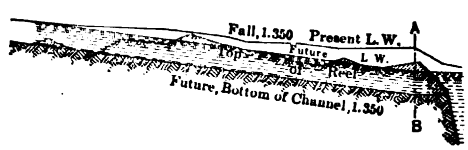


FIG. 41.

IMPROVEMENT OF THE DANUBE

in closing secondary arms, excavating cut-offs, and constructing revetments and levees, with the object of confining the river in low water to a single channel and preventing its overflowing the lowlands during floods. Dredging was largely employed to deepen the bed and thus avoid as far as possible any raising of the water surface when the works began to contract the natural flow.

Much work was done between 1885 and 1895 by the Hungarian Government on improving the section of 575 miles which flows through Hungary, and which in places was of difficult navigation, as is shown by the accompanying cut of the stretch near Vajka (Fig. 34). This work consisted chiefly of closing secondary arms, building training walls, dredging, etc., with the object of obtaining a channel 975 feet wide. Where the new channel was to traverse dry land, a small preliminary cut was excavated extending almost as far as the upper end, and dikes of earth heavily faced with riprap, to act as training walls, were built along the future limits of the stream. When all was ready, closing dikes were begun across the arms to be abandoned, and the river was gradually forced into the new channel, which it at once commenced to enlarge. Part of the mass of earth washed out was carried by the eddies through openings 150 to 300 feet wide, left in the training walls, while the remainder was disposed of by the river further down. At some places the current undermined the walls, but riprap was kept on hand to prevent serious injury. An illustration of the general arrangements is shown on Fig. 35.

The most noteworthy improvement, however, was carried out at the Iron Gates, a series of rapids and pools covering some 68 miles of river, and situated about midway between Vienna and the sea and on the frontier of Austria-Hungary. The name is stated by some to have originated in the extremely hard nature of the rocks, while others claim it to have been due to obstructions of chains placed across the channel by the Turks in mediæval times. There were five principal cataracts, covering an aggregate distance of some 5 miles, viz., Stenka, Kosla-Dojke, Izlas-Tachtalia, Jucz, and the Iron Gates, the last named being the furthest downstream (Fig. 36). This could only be passed by loaded vessels when the Orsova gauge read 8 feet, a stage which rarely obtained more than 116 days per annum. When the stage fell to 6.2 traffic was carried on by flat-boats, and at 2.6 all navigation ceased, and transfer by wagon was resorted to. Traces of a canal commenced by the Emperor Trajan are still to be seen at some points, and at various times attempts had been made to effect local improvement. The final work was begun in 1890 and completed in 1896, the estimated cost being about $4,000,000, which was to be repaid by tolls on navigation. The location of the principal points of improvement are shown by the black lines on Fig. 36. The mean width of this river in this section was from 1950 to 2300 feet, and the extremes from 512 to 6560 feet, the average fall being about 1.4 feet per mile. The local fall at Izlas-Tachtalia, over a distance of 5900 feet, was 1 in 760; at Jucz, over a distance of 2430 feet, it was 1 in 430; while at the Iron Gates the fall was 1 in 450 over a distance

of 6890 feet. At certain points the current ran from 7 to 11 miles per hour, shoals of a few feet in depth alternating with dangerous whirlpools where the water was from 100 to 160 feet deep. The low-water flow is given as 50,000 second-feet. The improvements consisted chiefly in the excavation of new channels with widths from 197 to 262 feet, and a least depth of 6½ feet. At the two upper cataracts the lengths of the channels, which were made straight, were 2620 feet and 8870 feet respectively, with widths of 197 feet, necessitating excavations of about 9700 and 86,300 cubic yards respectively. At the next section (Fig. 37), the promontory at Greben narrowed the low-water width to 690 feet, while just below the width became 6560 feet. The average fall was 1 in 1750. This stretch was improved by cutting a new channel about 1¼ miles long above the point, and constructing a dike 20,400 feet long below, whose crest was 6½ feet above low-water at the upper end, and 10 feet above at the lower end, with cross-dikes to the right-hand shore. This dike was estimated to contain 717,000 cubic yards of stone, while the rock excavation amounted to some 61,500 cubic yards. The maximum fall in the artificial channel was reduced to 1 in 940. At the Jucz cataract (Fig. 38) there existed a broad rock bar through which a channel was blasted 4265 feet long, necessitating the removal of some 42,000 cubic yards of rock. A dike nearly 2 miles long was built lower down as shown, in which were used more than 206,000 cubic yards of stone. The crest at the upper end was made 1.7 feet above low water, and at the lower end, 7.5 feet above.

The last and most formidable obstruction was at the Iron Gates, where at one point the main river plunged through a break in the reef 300 feet wide with a speed of 11 miles per hour and a maximum depth of 164 feet. This was changed by excavating a channel 262 feet wide by 9 feet deep and 6790 feet long, flanked by dikes 1¾ and 1½ miles long respectively, whose tops were 2 feet above high water (Figs. 39 and 40). The excavation for the channel amounted to 322,000 cubic yards of rock, while the stone in the dikes amounted to 847,000 cubic yards. The upper ends of the dikes were flared out funnel-wise, so as to ensure a full supply of water. The fall in the channel after completion was about 1 in 350 (Fig. 41), with a current velocity of 7 to 9 miles per hour. Steamboats ascend this without assistance, but other boats are pulled by cables.

The work was carried out partly in the dry, stone dikes being used as coffer-dams, and partly under water. For the latter ordinary submarine drilling was employed, and also rock-dredges which broke up the ledges by letting fall like a pile hammer iron rams or chisels weighing from 8 to 10 tons. The total amount of rock excavation was 1,635,000 cubic yards, of which 915,600 cubic yards were under water. The total cost was $6,696,000.

**Regulation of the Volga.**—There existed in European Russia in 1904 about 52,800 miles of waterways available for commerce, of which about 16,000 miles consisted of small streams suitable only for rafting or for downstream navigation. Of the first-named amount canals and canalized rivers comprised only 1220 miles, or 2⅓ per cent

of the whole. In northern Russia the season of navigation lasts from May to October; in middle Russia from April to November; and in southern Russia from March to November; the remainder of the year the rivers are frozen over. On the streams suited for rafting only, navigation may last for a month or less each year, while those of more importance usually become very low in summer, so that the majority of the traffic has to be carried on in spring and fall. During the rest of the year the boats lie idle.

The Volga, the Mississippi of Europe, contains in its basin 565,000 square miles, or nearly one-sixth of the continent, and includes 38 per cent, or more than 20,000 miles, of European Russia's navigable waterways, while 55 per cent of all the river steamboats and 37 per cent of other types of boats are found upon this stream or upon its tributaries. Owing to its size enormous cargoes can be carried, the average tonnage of the barges ranging from 1500 to 8000 tons, according to the class, with drafts up to 13 feet. The largest are known as "bielanas," and are built for downstream navigation only, like the coal boats of the Ohio, the craft being sold at the end of the journey for what they will bring. One of this type was built in 1903 for a cargo of 13,000 tons.

The country along the upper Volga is covered with deep snow from four to five months each year, and when the spring thaw comes sudden floods and heavy ice-gorges occur, often resulting in considerable damage to property and more or less change in the river's channels. The greater portion of the country traversed consists of agricultural land, but for several hundred miles above the delta vast expanses of salt-impregnated steppes are met with—the bed of an ancient sea—wholly useless for cultivation. The length from source to delta is about 2254 miles, with a total fall of 701½ feet, and the length navigable for boats, beginning at Tver, is about 1980 miles. At the source of the river are two storage reservoirs, with a total capacity of 19,395 million cubic feet. During the summer season these maintain a depth of 2 to 3 feet for 500 miles below. The low-water discharge at Tver, on the upper portion of the river, is 3900 second-feet, and the flood range about 37 feet. Near the mouth the minimum discharge is 113,000 second-feet, and the maximum, 1,426,000 second-feet, with a flood range at Astrakhan at the head of the delta of about 12 feet. The width of bed of the upper portion varies from 800 to 5600 feet at ordinary stages with a maximum flood width of 4 miles; the ordinary width of the lower portion, which extends for about 1650 miles from Rybinsk to the mouth, is from 2000 to 6500 feet, and the flood width, from 12 to 26 miles. Near the mouth the low-water width is about 9000 feet. The total fall in this portion is 319 feet, an average of 0.19 foot per mile. The bed in general is composed of alluvial matter, much of it being of fine sand. Owing to the gentler slope, however, the erosion of the banks and the changes of the bed are less sudden than in the Mississippi, and where the latter river would create a bar in a few days, the Volga would require as many weeks or perhaps months.

The improvement of the river was commenced with longitudinal and spur-dikes,

with the object of closing secondary arms and contracting the channels. Revetment was also largely used. The great length and consequent cost of the works, however—many of the spur-dikes were two-thirds of a mile long,—led to their gradual abandonment and to the adoption of dredging, supplemented by bank protection, and the methods of improvement have become identical in principle with those of the lower Mississippi. Below Rybinsk are about 370 bars which may need annual dredging (many, however, do not require to be dredged every year), the maximum depth required in low water being from 4½ to 7 feet, the minimum natural depth being about 2½ feet, although the depth in some of the pools at the same period may be 50 or 60 feet. The dredging will be more fully described in the next chapter.

**Regulation of the Mississippi.** (See Pls. 1*a* and 46).—The basin of the Mississippi, whose name, adopted from the Indians, signifies "Father of Waters," contains 1,256,000 square miles, equivalent to about 41 per cent of the area of the United States and to more than one-third of the continent of Europe, and comprises a length of 2500 miles in the parent stream and 15,000 miles in navigable tributaries. Three natural divisions are found, each of which has features differing somewhat from the rest. The first, the upper Mississippi, extends from the headwaters to the mouth of the Missouri, the largest tributary, a distance of about 1200 miles; the second or middle portion, from the mouth of the Missouri to Cairo at the mouth of the Ohio, the second largest tributary, a distance of 196 miles; and the third, or lower Mississippi, from Cairo to the Gulf of Mexico, a distance of 1073 miles. The upper portion has a comparatively stable bed and does not carry any unusual quantity of sediment. After its junction with the Missouri, which is heavily charged with alluvial matter, its character changes and the bed and banks, composed of this alluvial material, are easily eroded, causing a constant changing of local conditions. The lower portion displays the same conditions, but in an aggravated form, and comparative stability is not regained until the river has approached its delta. The slope of the upper portion below St. Paul averages 0.45 ft. per mile; that of the middle portion, 0.60 ft.; and that of the lower portion varies from 0.42 ft. for the first 300 miles to 0.1 ft. below New Orleans, the average being 0.25 ft.

As with the Volga, a system of artificial reservoirs, five in number, exists at the headwaters, with a total storage of 91,000 million cubic feet, sufficient to raise the low-water level at St. Paul from 18 inches to 2 feet. (See Chapter VII.) The minimum natural discharge at this city is given as about 1500 second-feet; the maximum, as about 117,000, and the extreme flood range is about 19.7 feet. The minimum and maximum discharges near the mouth are about 65,000 and 1,740,000 second-feet respectively, and the extreme local flood range in the lower river is about 53 or 54 feet.

In 1873, the improvement of the upper river below St. Paul was begun by a system of dikes, which was afterwards developed in connection with a general project for the regulation of the river from St. Paul to the mouth of the Missouri, a distance of about 658 miles. The original project contemplated a low-water modified depth of

4.5 feet, with a channel width of 400 feet for 30 miles below St. Paul, increasing to 600 feet for the following 27 miles, then to 800 feet for 155 miles, 1000 feet for 151 miles, 1200 feet for 158 miles, 1300 feet for 45 miles, 1400 feet for 70 miles, and to 1600 feet for the last 22 miles. These widths were determined partly by calculation, but principally by experience. The contraction was carried out almost entirely by the use of spur-dikes, aided by revetments at certain points, and as the natural tendencies of channel and curvature were studied with care and adhered to as closely as possible, the improvement proved generally successful. The maximum increase of depth was about 3 feet, and a navigable depth of 4.5 feet or more became generally available in such reaches except in seasons of extreme low water. The crests of the dikes, which were composed chiefly of brush and rock, were placed on a level with a 4-foot stage of water for about 470 miles below St. Paul, increasing to 5 and 6 feet lower down. In the section comprising the upper 30 miles were 29 bars, besides several shoals, and one stretch of 67 miles below the Chippewa River was a succession of sand banks, with little deep water between, there being 46 bars which caused special obstruction. The width of 800 feet adopted for the channel in this section was reported to be too liberal for the great amount of sand to be scoured out.

In 1907 the project depth was changed from 4½ feet to 6 feet, to be obtained by additional contraction of the channel aided by dredging. The channel width at St. Paul was reduced to 300 feet, increasing gradually to 1400 feet at the mouth of the Illinois River, and retaining the latter width as far as the mouth of the Missouri. Through the rapids at Rock Island the width was to be from 200 to 250 feet. The cost was estimated at $20,000,000, and the annual cost of maintenance after completion at $300,000. The improvement was to be completed not later than the year 1927.*

Two improved reaches of the Upper Mississippi are shown on Pl. 6.

The methods employed for improving the middle portion, below the mouth of the Missouri, as far as the mouth of the Ohio, about 196 miles in length, were very similar, except that revetment had to be employed to a much larger extent, and that dredging was and is extensively used to create and maintain the low-water channel. The depth desired from the Missouri to St. Louis was 6 feet, and from St. Louis to the Ohio, 8 feet, the original low-water depths having been from 3½ to 4 feet. The width of the contracted channel was made about 2500 feet, and the dikes were of the permeable type, longitudinal or spur-dikes being employed according to the need of the case. Estimates made in 1910, looking to the completion of the work by 1922, gave the total probable cost from 1872 to 1922 as approximately $30,000,000.

The lower river, however, from Cairo at the mouth of the Ohio to the sea, is the distinctive portion of the Mississippi.† The valley ranges in width from 30 to 60 miles,

* For a description of the project and of the methods of work, see "Engineering News," May 6, 1909, and April 11, 1912.

† A general map of some of this portion will be found at the end of the book, on Pl. 1*a*. Details of the dredging operations will be found in the next chapter.

and before the building of levees an area of some 30,000 square miles was subject to inundation by high floods to a minimum depth of 5 to 10 feet. The high bluffs which form the limits of the bottom-lands are composed of strata of sand, clay and gravel, more or less compact, but yielding to a prolonged attack by the river. The bottom-lands themselves are of light alluvial soil, easily eroded under exposure to strong currents. It is estimated that the annual erosion below Cairo amounts to an equivalent of 10 square miles 86 feet deep, which is many times more than the whole volume of sediment carried to the sea, as the material cut from one locality is dropped in slower currents downstream. (See p. 23.) The amount of sediment varies from 1 in 400 to 1 in 6000 by weight, and the total amount carried annually by the river from its basin to the Gulf is estimated at 518,500,000 cubic yards. The banks, as with all alluvial streams, slope away from the river, the fall in the first mile at right angles to the river being about 6 feet (in occasional places reaching as much as 10 feet or more), and then gradually decreasing. The width of the bed in low water averages about 2600 feet in the bends, with depths which may reach 100 feet or even more. Near and below New Orleans, after the bed and banks have attained the comparative stability which first becomes noticeable near Baton Rouge, 240 miles from the sea, the width becomes about 2000 feet, expanding to about 7000 feet at the head of the Passes, and low-water channel depths of 90 to 120 feet are found for stretches of many miles, with occasional bends where the depth reaches from 180 to 200 feet.

The average low-water channel depths in the bends are 18 feet from St. Louis to Cairo; from Cairo to Memphis, 31 feet; from Memphis to Vicksburg, 37 feet; from Vicksburg to the Red River, 48 feet; and from the Red River to New Orleans, 84 feet. In New Orleans is one bend with a maximum low-water depth of more than 200 feet. Near it a few years ago a 5000-ton steamship capsized and remained sunk without interfering with traffic.

The history of the improvement of the lower Mississippi is of unique interest to the engineer. While other rivers, such as the Volga or the Paranà, approach its volume of discharge, and others still, such as the Ganges or the Indus, far surpass it in their ceaseless shifting, no systematic improvement of any river has yet been undertaken which will be so far-reaching in its results, or where the engineers have had to cope with forces like those of this giant stream. The construction of the levees, by means of which thousands of square miles of fertile land were reclaimed; the experiments for improving the channel by means of dikes, which proved unequal to the contest unless built of a type whose cost would be prohibitory; and the consequent gradual adoption of dredging, form records of great interest. The operations for improvement, exclusive of the levees, are now limited to the revetment of banks at the most important (though isolated) points, and the maintenance by dredging of a low-water channel 250 feet wide and not less than 9 feet deep.* Descriptions of this work will be found in succeeding chapters.

* About 1908 this depth was experimentally increased to 14 feet, at occasional points, and with success, in connection with a proposal for a deep-water route between Chicago and the Gulf of Mexico.

The saving per ton on the 4,550,000 tons of freight, including boats and rafts, actually passing over the upper Mississippi in 1903 was stated to be $876,650, the average freight rate being $2.83 per ton as against $3.48 per ton charged by competing railroads.

**Regulation of the Missouri.** (See Pl. 46.)—The regulation of the Missouri, the largest tributary and in fact a longer river than the Mississippi, is also of great interest, as indicating the possibilities of controlling a river of great variation of discharge and of pronounced alluvial characteristics. This stream, whose name in the Indian language signifies "The Great Muddy River," drains 580,000 square miles, its extreme length being about 2945 miles. The head of ordinary navigation is at Fort Benton, 2285 miles above the mouth, and in connection with the Mississippi the waterway extends without a break for some 3500 miles to the Gulf of Mexico. The upper 430 miles flow through a stable bed, but about 170 miles below Fort Benton the alluvial features begin to appear, and from Sioux City to the mouth, a distance of 807 miles, they are very pronounced. The width of the valley in this portion varies from $\frac{3}{4}$ to 17 miles between its limiting bluffs, and rock is found at depths of 70 to 100 feet below the general level, the valley appearing to be a vast trough filled with varying qualities of sand and gravel. The flood range is from 19 to 35 feet according to locality, and the discharge at the mouth varies from about 15,000 to about 900,000 second-feet. It is estimated that there is at all times a flow sufficient to maintain an improved channel with a depth of not less than 8 feet near the mouth, and not less than 5 feet at Sioux City. In the unimproved portions, however, the low-water depth on the bars rarely exceeds 3 feet, and the width of the river at those points varies from about $\frac{1}{3}$ of a mile to 1 mile. In deep reaches the width varies at low water from 800 to 1500 feet, with depths often somewhat in excess of 12 feet. In high water the depth on the bars is about 9 feet only, as the latter build up very rapidly during a flood. The average slope below Sioux City is very uniform and is 0.86 foot per mile, the minimum being 0.70 foot and the maximum 1.12 feet. As might be expected with a river of this slope, the erosion is excessive, and a vast quantity of sediment is taken up and transported, the annual amount carried out of the river being estimated at 413,000,000 cubic yards, while the proportion by weight varies from 1 in 2000 at mean low water to 1 in 200 in floods. The bed is consequently tortuous in many places, and uncertain; bars form rapidly, and the channel displays the usual contrasts of width and depth, and is often divided by islands and marked by severe erosion of the banks.

The improvements began in 1876, with the object of creating and maintaining a single stable channel. Experiments of many kinds were tried before methods were evolved suited to the peculiar conditions, and the final one adopted involved principally the use of permeable spur-dikes of piles, supplemented by bank revetment. The latter is considered to be of the first importance, as from the caving banks the river obtains most of its material for forming bars. Owing to lack of funds and to the decision of Congress to restrict the work, only one continuous portion of the river could be improved.

This was a stretch about 45 miles long, commencing about 65 miles from the mouth, and extending upstream. The channel width was made from 1000 to 1240 feet, and the depth was increased by the scour from the original $2\frac{1}{2}$ feet to 6 feet. This had been one of the worst sections of the river, but the regulation not only produced an excellent channel, but resulted also in reclaiming an amount of land whose value went far to offset the cost of the improvement. In addition to this work, a considerable amount of dike and revetment construction was carried out elsewhere in order to protect local points of importance. This will be described in other chapters.

The economic history of this river is of great interest. For more than 200 years it formed the link of communication between the Mississippi and the Northwest, and during the nineteenth century it became the chief trade-route for the western country. The hunters and trappers, the pioneer settlements, the exodus of gold-seekers to California in 1849, the subsequent campaigns against the Indians, and the final civilization of the Great Plains, all followed the course of this great stream. The commerce upon it grew to be, for that day, of immense proportions, and so continued until the advent of the railways, when the traffic began to forsake the interruptions and dangers of the unimproved river for the perennial and quicker services by land. Before 1880 the river commerce had practically gone, and it was not until its abandonment seemed inevitable that the inhabitants of the valley began to make determined efforts to have the stream improved so as to make traffic once more a commercial possibility. The condition of affairs was very similar to that of the Rhine about the same period; in each case the railways were stifling the river traffic, and would soon be relieved of its competition unless a better waterway could be evolved. The Rhine, however, flowed through a populous country; its 570 miles of navigable water were already largely improved; and it had its outlet at a seaport. The country of the Missouri was thinly settled and many of the most important towns lay off its course; its navigable length was more than 2200 miles, and little work had been done upon it; and its mouth was a thousand miles from any sea. The appropriations made for the improvement were but a fraction of what would have been needed to regulate the river and change it to a modern waterway, and during the twenty odd years of the existence of the commission to whom the expenditures were entrusted, the members were obliged to spend much of the money under pressure from local and isolated interests, and could succeed in completely regulating only the 45 miles before alluded to. The inhabitants of the valley, beginning to realize in part the magnitude of the work they had required, and finding that a generation or more must elapse under existing appropriations before definite benefit would ensue, gradually withdrew their support, and in 1902 Congress decided that further improvement of the river should be abandoned. Ten years later, however, a first appropriation was made towards obtaining a permanent 6-ft. channel between Kansas City and the mouth, and it seems certain that as the population and the wealth increase, this great river will play a part again in the development of the West.

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**Rhone.**—The regulation and maintenance from 1871 to 1896 of the 206 miles between Lyons and the sea cost a total of $12,670,000, or about $61,500 per mile; of this amount maintenance cost $2,065,000 and new work $10,506,000, or about $9500 and $52,000 per mile respectively. Maintenance alone in 1896 reached a total of $160,000, or about $780 per mile. The depth from Lyons to the tidal portion was to be $5\frac{1}{4}$ feet, and in the latter, $6\frac{1}{2}$ feet. Permanent materials were used for all dikes.

**Loire.**—The cost of the establishment of training works is given as $32,000 per mile.

**Mississippi.**—Estimates made in 1904 for completing the regulation of the 658 miles between St. Paul and the mouth of the Missouri, and including the cost of all work done up to that time, gave a total of $27,500,000 for a depth of $4\frac{1}{2}$ feet, and $35,000,000 for a further contraction to secure a depth of 6 feet, equivalent to about $42,000 and $53,000 per mile respectively. The annual cost of maintenance of the completed work was estimated at $380 per mile. The dikes used were composed of piles, brush, and stone.

**Missouri.**—The cost of regulating a difficult stretch of 45 miles near the mouth, in order to obtain a depth of not less than 6 feet, was $58,000 per mile. This amount included the cost of a large plant and many necessary expenditures for experimental work. The dikes were made of piles and other non-permanent materials.

* From "Rivières à Courant Libre," and various Government reports.

# CHAPTER III.

## DREDGING AND SNAGGING.

NOTE.—Certain of the illustrations are reproduced by permission from papers by Mr. J. A. Ockerson and Mr. A. W. Robinson; and others are furnished by the Bucyrus Co., South Milwaukee, Wis.

**Evolution of Dredging.**—The fact that it may cost too large a sum of money to render a river satisfactorily navigable by the best methods often leads to the acceptance of makeshift or temporary forms of improvement. The temporary contrivances used for this purpose, or suggested for trial, are of great variety, and consist of trellis-work, basket-work, movable gates, swinging mattresses, etc., for deflecting the current, and of scrapers and stirring devices of various designs for loosening the bars and assisting the current in their removal, etc.

The principal trouble with all these appliances, however, arises from the fact that the current cannot be relied upon to carry away sufficient material after it has been loosened, nothwithstanding a general belief that when the crust of a bar has been broken it is an easy matter to wash it away. Janicki has called attention in these words to one effect produced by them: " Such machines have a moral effect, if I may be allowed the term. Boat-owners, who have to suffer from low water on the bars, complain less if they see that something is being done to relieve them." The principal of these devices are as follows:

*Gates.*—M. Borrel and M. Collin * applied upon the Garonne and the Loire, respectively, more than a half century ago, movable gates or planks which were placed so as to contract temporarily the current over a bar and produce scour by the increased velocity. Thus a passage for boats was created by a simple displacement of material. The gates were reset as often as required. This device is still in use in France, and has been tried also on certain parts of the Mississippi and in India. (See also Chapter IV.)

M. Fouache used a trapezoidal leaf having teeth along its lower edge to deepen the Somme Canal. It was supported at the desired height by boats, so as to confine the stream and thus increase the water-level above and reduce it below. The head thus produced moved the gate along the canal, and with it the material torn up by the teeth as the leaf traveled. The depth of the cutting-edge was regulated by windlasses on the boats.

Engineer Masquelez, in 1811, used a similar device in building the canals at the mouth of the Charente. It was placed astern of a suitable boat, the bottom being

* Annales des Ponts et Chaussées, 1835.

loosened with a hook. Two movable wings served to close the canal behind the boat and thus produced the head necessary for movement, the bottom of the canal being cut as with a plane and pushed along toward the outlet.

*Screws.*—" About the earliest application of this principle on the Mississippi was in 1867, when it was decided to improve Pass à l'Outre by means of excavating and stirring up the alluvial material deposited from the heavily laden waters of the river.* In this work a double-ended dredge-boat, having an excavating-screw with four blades 14 feet in diameter, was used. This screw was similar to an ordinary propeller-wheel and was similarly mounted. It was turned by means of a double engine at the rate of 60 revolutions per minute, and reached a depth 2 feet below the keel. The work of the screw was made more effective by auxiliary scrapers attached to the upstream end of the boat, on each side of the keel. The boat was moved downstream over the bar with the screw operating and the scrapers in position. In this way some of the bar material was again brought into suspension and carried off into deep water by the current.

" During the first month's work with this dredge the depth was not materially improved. Later, better success was realized, and in a little less than two months the depth was increased from 11 to 17 feet. The chief difficulty seemed to be in weak propeller-blades, which were frequently broken and could only be renewed by docking the vessel. This device was intended to cut out and maintain a 20-foot channel through the bar at the mouth of the river.

" At the Southwest Pass the same result was expected from the use of conical screws attached to the bow of a suitable boat. These cones were 20 feet long and 5 feet in diameter at their bases. They were set so that their points came together at the boat's stem, and their bases were separated so as to cover a width of 20 feet from out to out. Their axes were horizontal, the salient angle being foremost. The flanges of the screws were 12 inches wide at the base of the cones, and diminished to 6 inches at the points. When these enormous screws were put in motion it was very difficult to guide the boat. The material was readily plowed up, but it was not broken sufficiently fine to be carried away by the current.

*Scrapers.*—" In 1867 there was appropriated the sum of $96,000 for the construction and operation of two scrapers or dredges on the upper Mississippi, between St. Paul and the mouth of the Illinois River. The first efforts made to remove the sand-bars by means of the scrapers, which were invented by Col. Long, was in the fall of that year. These scrapers consisted of a frame attached to the bow of a boat and carrying a heavy cross-bar, to which were attached six steel buckets or cutters. The frame could be raised or lowered at will. In operating, the boat went to the upper side of a reef, the scraper was lowered, and the boat was backed slowly down stream, scraping the sand with it to the deep water below the reef. This operation was repeated until the desired depth was obtained. Two side-wheel steamboats were equipped with these

* Trans. Am. Soc. C. E., vol. xl, pp. 223–225, J. A. Ockerson.

scrapers by the Government, and, for a time, steamboat owners operated a scraper boat between Keokuk and St. Louis at their own expense.

"One boat was equipped and ready in October, 1867. Her first work was on a bar near Gray Cloud, 17 miles below St. Paul. Only $3\frac{1}{2}$ feet of draft could be carried over this bar, and the regular packets could not cross it. After about four hours' work with the scraper the depth was increased to $3\frac{1}{2}$ feet entirely across the bar. The scraping was continued for two days and a depth of 4 feet was secured. By November 15th all the bars between St. Paul and Prescott had been scraped and the depths increased to $3\frac{1}{2}$ or 4 feet. At that date the packet companies notified the engineer in charge that the scraping had removed all obstructing bars and that no more work was required.

"In 1868, when navigation again became difficult, the scrapers were put into commission and worked throughout the season. They succeeded in deepening the bars from 8 to 18 inches, and this was generally accomplished with a few hours' work. Beef Slough was deepened from $3\frac{1}{2}$ feet to $4\frac{1}{2}$ feet in thirty-five minutes.

"On the whole, the results were so satisfactory that steamboat owners announced that their boats had been making regular trips without interruption, 'a condition of affairs never before known at this stage of river in the experience of pilots of thirty-five years' standing.' The largest steamers had been able to reach St. Paul in the low-water season during two successive years, when without the aid of the scrapers they would have been obliged to tie up.

"This scraping was continued for several years at a cost of about $20,000 per annum for each steamboat, but, as the relief was only temporary and had to be repeated from year to year, it finally gave place to the so called-permanent improvement, consisting mainly of channel contraction.

"It should be borne in mind that in the portion of the river where the above-described scrapers were used the obstructing sand reefs are quite short."

*Jets.*—In the various inventions brought forth the water-jet has had a prominent place, the object being to enable a steamboat, upon running onto a bar, to work its way through by means of pumps, and some of these devices have met with considerable success. In 1881, for example, a bar near St. Louis was cut through by the use of pumps mounted on boats, and having a capacity of about 165 gallons each per minute. In ten hours the channel was deepened from 6 feet to 8.3 feet, and made wide enough for the largest tows.

A special jet-dredge was built in 1896, for use on the Mississippi between St. Louis and Cairo, the pumps being two 15-inch centrifugals, each with a capacity of 10,000 gallons per minute. On short bars the work was very effective, but on long ones the piling-up of the sand in front of the jets prevented successful results.

**Dredging.**—Dredging as a means of river improvement has been, and is being increasingly employed in all parts of the world, and the extent of its application has kept pace with the evolution of the powerful modern dredges which began to appear after the

year 1890. While its effects are often temporary, the work having to be repeated occasionally and sometimes constantly, it has secured and maintained in many cases an improvement at much less cost than could have been obtained by jetties, dikes, or other permanent works, and it has become established as a very important feature of engineering. It forms, for example, the principal method used for improving the navigability of the lower Mississippi and of the rivers of Russia, especially the Volga and its tributaries, the Oka and the Kama, as described further on in this chapter.

**Location and Size of Channel, etc.**—This will be found discussed on page 64 and the pages following it.

**Types of Dredges.**—The following are the principal types of dredges:*

*Dipper Dredge* (Figs. 42 and 44).—This consists of a boat provided with an iron dipper or bucket, secured to the end of a long handle which is mounted on a revolving boom so as to permit a wide range of digging. For operation the dipper is lowered to the bed of the river, and pulled along in an almost horizontal direction till it has scraped up a load. It is then raised (the handle being arranged to slide up and down and held wherever desired by friction), the boom is swung, and the load dumped into scows or elsewhere by opening the hinged bottom of the dipper. The boat is held in position by " spuds," or long timbers, which move vertically in sockets in the hull, and rest on or penetrate the river bottom during the digging. Three spuds are provided, one on each side of the bow, and one at the middle of the stern. For digging in hardpan or similar substances, large teeth are bolted to the mouth of the dipper, with the object of breaking up the material.

These dredges are at present made of sizes from ½ to 15 cubic yards in capacity and to operate in depths of water up to 50 feet, the largest up to a recent date having been used on the improvement of New York Harbor. The dipper had a capacity of 15 cubic yards, and a clam-shell bucket of 12 cubic yards could also be used. The limit of depth in digging was 50 feet, and the average speed in water 45 to 48 feet deep was one dipper per minute. This dredge has a record of 6000 to 8000 yards per day.

The dipper type of dredge is by far the most useful for general work, as it can be used for dredging, for tearing up old foundations, for making embankments, for lifting heavy weights, and in similar capacities. It requires a considerable amount of space in which to work, owing to the method of operation, and in this respect it is inferior in certain situations to the clam-shell dredge. It will, however, excavate almost any material except solid rock. The speed of working varies under favorable conditions from 30 to 60 seconds per dipper load, according to the depth. The limit of economical depth under ordinary conditions is from 30 to 40 feet, owing to the great size of the parts required for deep excavation and to the strain on the spuds. If the latter are made of sufficient strength to preclude breakage they may become very expensive.

* Tables of sizes, etc., of the different types will be found at the end of this chapter.

In one or two cases they have required a size 36 inches square and about 60 feet in length. For deep dredging the clam-shell type is usually preferable.

*Clam-shell Dredge.*—This style is similar in general arrangements to the dipper dredge, but has a clam-shell bucket instead of a dipper. This is shaped like a semi-cylinder, hinged so it will open into two parts. It is suspended from the end of the boom, being kept from twisting by two long poles which work up and down through eyes on the boom, or by a weighted line fastened to one side, and running through a block. It is operated by two chains, one of which lowers the bucket rapidly in an open position so it will penetrate the material, while the other is arranged to pull the halves together, scraping up a load, and then hoisting the bucket to the surface. The load is dumped by slackening this chain and holding the other, when the bucket opens again as in its first position. Frequently a hemispherical shape, divided into four parts, is used instead of a semi-cylindrical one divided into two; the bucket is then called an "orange-peel" or a grapple. This style is more serviceable for general use than the other. The speed of working is about the same as that of the dipper dredge.

Formerly a type was made with only one chain, which pulled the parts together, latches being employed to release the load. It was found, however, that when the blades caught on any immovable substance, such as a buried log or a projecting rock, it was almost impossible to get them loose, as the chain only acted for closing, and for this reason the type was abandoned. Recent modifications in design are stated to have overcome this objection, and to have produced a type which operates satisfactorily.

The clam-shell dredge of ordinary size will only operate effectively in material such as gravel, blasted rock, etc. For hard or tenacious clay it is of little value unless a very heavy bucket is used. It has the advantage, however, of being able to work in confined positions, such as in sinking caissons, and also of being suited to great depths, as there is little straining on the spuds, such as occurs with the dipper dredge. It can also use a much longer boom.

*Combination Dredge.*—For miscellaneous work the dipper and clam-shell type are sometimes combined, the boom and machinery being so arranged that either style of bucket can be used. This is sometimes very convenient for river work, especially in dredging in lock-chambers. Another type of combination dredge, met with principally in Europe, consists of an elevator dredge discharging into hoppers in its hull, from which the material is removed by a centrifugal pump and placed on shore through pipes.

*Elevator or Ladder Dredge* (Figs. 43 and 45).—The type generally employed in England and other parts of Europe is that known as the elevator, ladder, or bucket dredge. It is used in Canada also, although it has not met with much favor in the United States, as it is expensive in first cost and in operation, and has not the adaptability for general work such as a contractor requires. With it tolerably hard materials may be excavated to a considerable depth. It consists of two parallel endless chains, carrying a number of buckets which travel along an arm swung at the upper end from a frame, while the

FIG. 42.—General View of a Two-yard Dipper Dredge, with Wire-rope Hoist. (See also Fig. 44.)

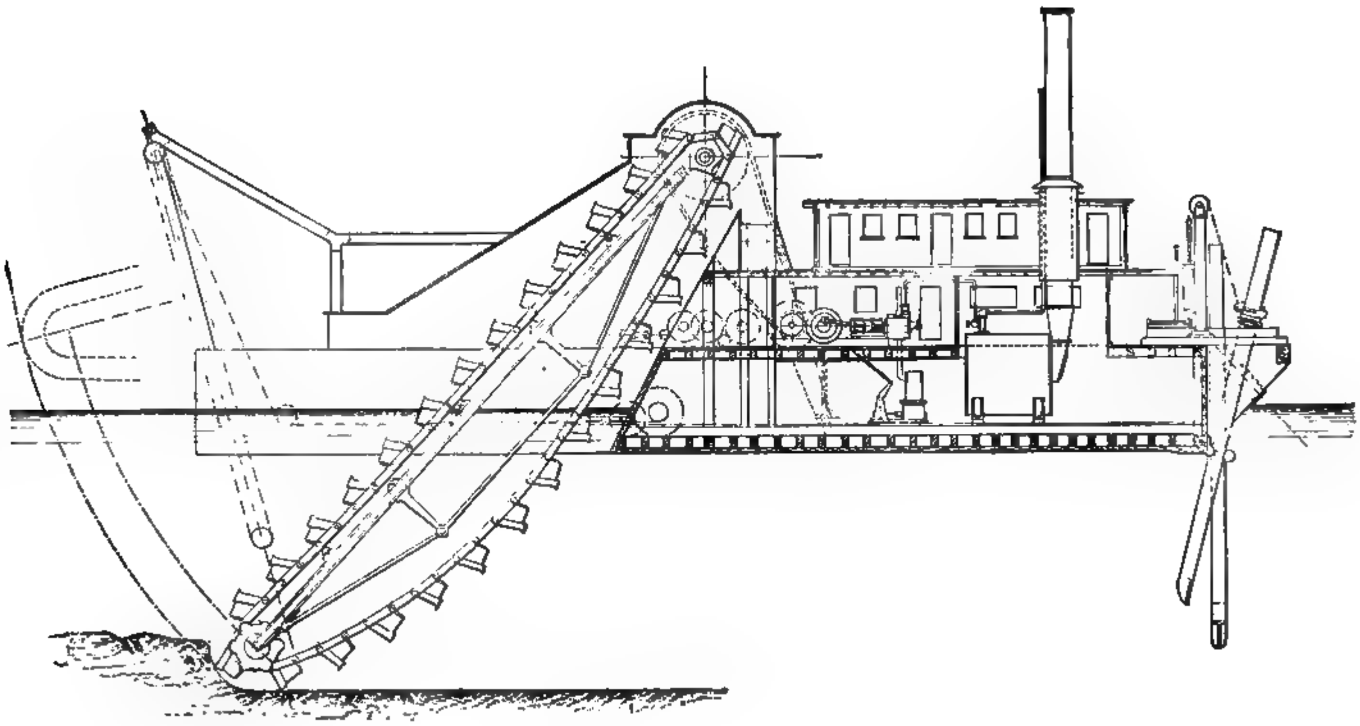


Fig. 43.—Section of an Elevator Drdege.

Fig 44.—General Plan, etc , of a Two-yard Dipper Dredge.

lower end rests on the river-bed. The chains pass around drums at the two extremities of the arm and are driven by powerful machinery. The arm may be adjusted to suit the depth of water, etc., and may be vertical or inclined. As the material is brought up it is discharged into a hopper or into a chute, sufficient water accompanying it to wash it to the point desired, or to load it into scows. Some of these dredges are arranged for self-propulsion, and the material is then dumped directly into a hopper in the boat itself. When this is full, dredging is stopped and the boat is run to the dumping-ground and the material deposited. This method has the advantage of causing no lost time, the same crew doing all the work, but it is of course not as speedy as when dredging goes on constantly, with a separate crew attending to the deposit of the material. In working, the dredge is moved along slowly in order that each bucket may be filled. The speed varies from 10 to 18 buckets per minute, and the capacity of the buckets, for general work, from 11 to 15 cubic feet apiece, although in soft material they can be used thrice this size. Usually the bucket-ladder is located on the center line of the hull, but in some cases it is placed at the side. The type is also built with two separate sets of buckets.

Two dredges of the elevator type were used on the lower Mississippi in 1888, and did very efficient work. One of them is reported to have dredged 4000 cubic yards in a ten-hour day.

*Rock Dredges.*—There was used in France as far back as 1852 a special type of dredge for excavating rock. A somewhat similar apparatus was used on the Suez Canal, and later at the rapids of the Mississippi at Rock Island, at the Iron Gates of the Danube, on the Rhine, the Hudson, and elsewhere. The machine has a series of chisels or pointed rams, about 8 inches square and 16 feet in length, and weighing from 4 to 10 tons, which are run in leads and let fall from a height on to the rock. Behind the chisels in some of these machines is arranged an endless chain with buckets for removing the rock when broken. In others the ram is separated from the dredge proper, the one cutting and breaking the rock into pieces while the other removes it.

In the improvement of the Rhine the rocks in certain portions of the stream were drilled by using an oval-shaped diving bell (of 16 feet by $24\frac{1}{2}$ feet horizontal dimensions), which was framed into the hull of a boat, and could be raised or lowered to suit the depth. The maximum depth attainable below the hull of the boat was $16\frac{1}{2}$ feet. Before blasting the boat and bell were removed to a distance of about 200 yards.

*Hydraulic Dredges* (Fig. 44*a*, 46, etc.).—This type of dredge, sometimes called a "pump dredge," which has come into general use in other countries also, has practically superseded other types in the United States for all work of extensive nature in soft material, and is frequently used on work of minor importance also because of the convenience of disposal of the excavation. The material is sucked up by a centrifugal pump through a pipe whose mouth touches or lies close to the bottom, where the rush of water draws in the particles, discharging them either through another pipe for a distance which has reached at times more than 12,000 feet, or, in the case of a hopper dredge, into hoppers

or bins in the boat. Three general types are in use, the oldest being the one used for dredging in rough water, as on ocean bars, and in which the suction pipe is dragged or pushed along the bottom by the boat working to an anchor, or by using its own propelling machinery. The motion scoops up the sand and forces it into the pipe. In the second type, to which belong the Mississippi dredges, the sand is stirred up by a series of jets discharging close to the opening of the suction pipe, and worked by a special pump on the boat. The last type, the evolution of the other two, is the cutter dredge (Figs. 46 to 53), in which the material is loosened by a series of knives shaped so that they will not clog, fastened to a shaft and revolving close to the end of the suction pipe. While the first two types have proved to be of little use in any material more compact than sand, the cutter dredge will work successfully and rapidly in material as hard as compact clay, and even in sand has proved considerably superior to the drag and the jet method.* For this reason it is preferable under ordinary circumstances, because in most excavation the class of material varies, and where a seam of clay would render the jet dredge practically helpless, the cutter dredge will continue work unhindered. The cutter is mounted on a " ladder " or frame which is raised or lowered as needed, and is driven by an engine on the frame or by a bevel gear connected to an engine on the boat. The ladder, where work has to be done in shallow water, is usually placed projecting from the end of the boat, and must of course project far enough to allow the excavation to be completed to the side of the cut before the hull touches the bank when swinging. To facilitate this the ends of the hull should be curved. Two spuds are generally used at the end opposite to the ladder, one on each side, and they are alternately raised and lowered, " walking " the boat forward as she is swung back and forth by side cables. The discharge pipe, usually made in lengths from 30 to 100 feet, and used in occasional instances as large as 42 inches in diameter,† is supported if above water by wooden or steel pontoons, and if in the water by other lengths of pipe with closed ends. Flexibility is secured at the joints by short pieces of heavy rubber tubing, or, a method often employed in Europe, by joints of leather.

The suction dredge possesses one great advantage over the dipper or the elevator type in that it can discharge directly and at long distances into the water or on shore, and is therefore independent of scows or tugboats. If provided with specially flexible pipe joints it can also work in moderately rough water, and it is stated that work has been done with waves running from 5 to 6 feet in height.‡

In certain hydraulic hopper dredges, the hopper bottoms are arranged with a double row of doors. The space between these doors, which can be connected with the pump at one end and with the water at the other by means of valves, can be used as a suction

* A 20-inch hydraulic dredge was employed in 1910–1912 in the Hawaiian Islands with a cutter designed to remove coral rock, and is said to have excavated about 4000 cubic yards of this material in 24 hours. For description see "Engineering News," June 13, 1912.

† A hydraulic dredge (without a cutter) was employed in 1905 at Galveston with a 60-inch discharge line. It was brought from Holland.

‡ International Congress of Navigation, 1898.

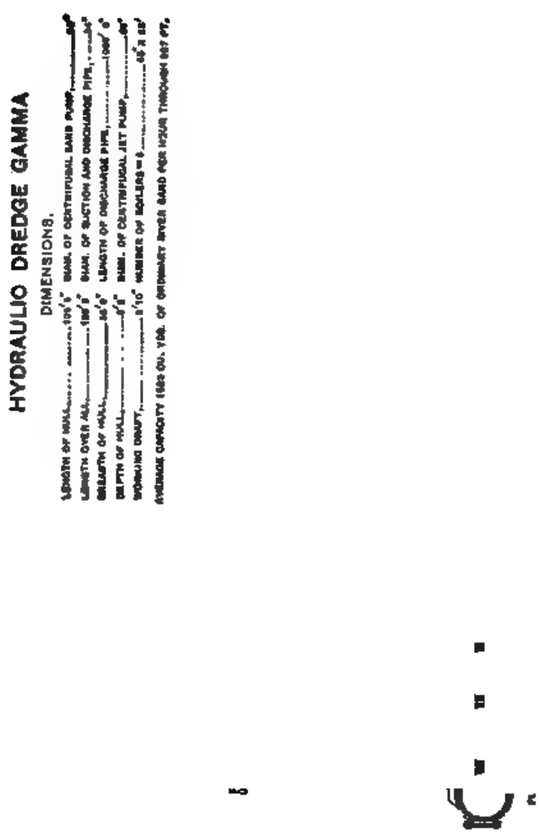

HYDRAULIC DREDGE GAMMA
DIMENSIONS.
LENGTH OF HULL
LENGTH OVER ALL
BREADTH OF HULL
DEPTH OF HULL
WORKING DRAFT
DIAM. OF CENTRIFUGAL SAND PUMP
DIAM. OF SUCTION AND DISCHARGE PIPE
LENGTH OF DISCHARGE PIPE
DIAM. OF CENTRIFUGAL JET PUMP

FIG. 44a.

FIG. 45.—General View of an Elevator Dredge Arranged for Shore Discharge. (See also Fig. 43.)

pipe by gradually opening the upper doors and letting the material fall through into the water as it passes to the pump. This method is frequently used to discharge on shore material brought in the hoppers from a distance. Another useful feature is the introduction of overflow valves in the hoppers, sometimes at three different heights, so that the hoppers can be loaded to part or full capacity, according to the depth of water in which the boat must work.

A very convenient type of hydraulic dredge for general work has been found to be one of the cutter type, with the suction pipe not over 20 inches in diameter, while for small work one with a 12-inch pipe has proved well adapted. Plant of larger capacity can only be used economically on very large work. It must be noted, however, that cutter dredges can rarely be used to advantage in exposed situations, as any pronounced movement caused by waves results in severe strains on the spuds and on the ladder. Cases are on record where spuds 24 inches square have been thus broken off. The drag-suction appears to be the only type which can be safely used in such localities.

An excellent example of a cutter dredge, the "J. Israel Tarte" (Figs. 47 to 52), was built in 1902 for the Canadian Government, for use in deepening and widening the ship channel of the St. Lawrence River between Quebec and Montreal. This dredge established in 1903 what was up till then a world-record for a month's dredging, 600,000 cubic yards (place measurement) being removed in 25 working days, equivalent to 1320 cubic yards per hour. During this time she was in operation 83 per cent of the full working time.* This dredge works sideways, and can reach a maximum depth of 50 feet, discharging through a 36-inch pipe to a distance of 2000 feet. The pipe-sections are 100 feet long, as this length was found preferable to lengths of 50 feet in rough water, and the joints are arranged to stand the effects of waves 5 feet high. The pump engine is triple-expansion, running at 150 revolutions per minute with 150 pounds of steam, and the cutter engine is of 300 horse-power. The cutter itself is 9 feet 6 inches in diameter and 9 feet long, weighing 10 tons. Quarters were provided for 36 men, 18 men being employed to a shift. The contract price of the dredge was $163,800.†

A 20-inch hydraulic dredge, working under ordinarily favorable conditions and in material not too hard, can average from 250,000 to 500,000 cubic yards per month. This includes the time for ordinary stoppages and repairs, and supposes that 3 shifts of men are employed per 24 hours. One of the 20-inch hydraulic dredges used on the construction of the New York State Barge Canal exceeded 500,000 cubic yards per month, and averaged 885 cubic yards per hour of actual working time during October, 1910. It is stated that records have been made in harbor work under exceptionally favorable circumstances, of more than 700,000 cubic yards per month.

The accompanying cut of the "Atlantic" (Fig. 53) shows an unusual type of cutter dredge, and it is to be noted that the ordinary barge hull has been abandoned in favor

* Transactions Can. Soc. C.E., A. W. Robinson.

† Additional data will be found under the head of "Cost of Dredging."

FIG. 46.—Twenty-inch Hydraulic Dredge with Cutter, Built in 1910 for the Mohawk River Improvement, New York State Barge Canal.
(See also Figs. 44a, 49, etc.)

of a sea-going hull. This was done in order that the dredge could be sent from place to place without the risk of being sunk if caught by rough weather while in transit, an accident which has befallen several of those built with the usual type of hull.

FIG. 47.—View of Bow.

FIG. 48.—View of Pipe Line and Stern of Dredge.

The development of the cutter type has been accompanied by a similar development of the drag-suction hopper dredge (Figs. 54 to 56), since, as just noted, the former are unsuited to work in the open sea. The first of the latter type in this country was built about 1855; but it was not until some thirty-five years later that the utility of these

dredges began to be recognized. Since 1890 a considerable number have been built, and they are now to be found at all important points along the coast of the United States, and at many foreign harbors. While their work is naturally of a temporary nature, they have proved very efficient, and have enabled entrances to sea-ports to be maintained where the cost of improvement by jetties would have been prohibitive.

The largest dredges of this type are believed to be those employed at the entrance of New York Harbor and at the mouth of the Columbia River in Oregon. At the last-named place, owing to the rough sea, a large hull was especially necessary, and the gross tonnage of the dredge used was 5590 tons. She was originally used in the Atlantic trade as a merchant ship, being purchased later for an army transport, and finally transformed into a dredge. A large boat is much more effective than a small one for exposed work,

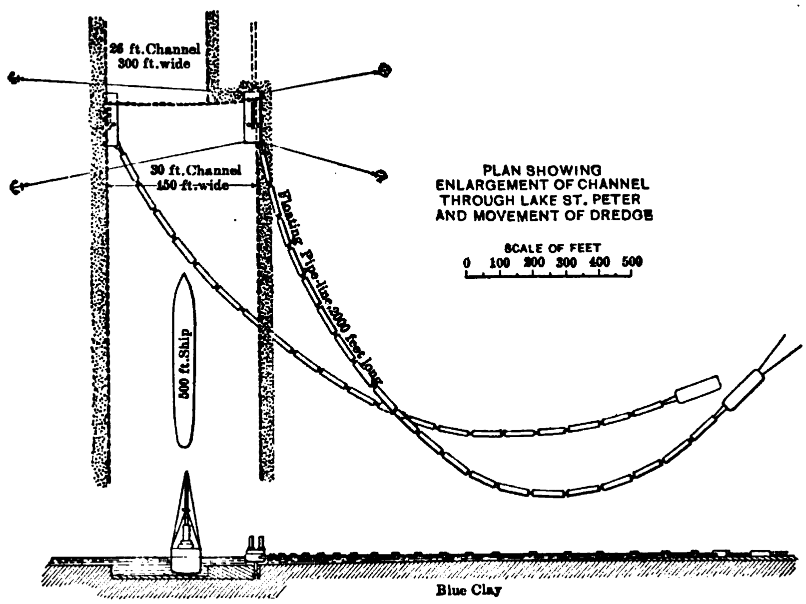



FIG. 49.—View of a Hydraulic Dredge in Operation. (The dredge shown is the "J. Israel Tarte," at work in the St. Lawrence River, Canada.)

since it can continue operations in weather which would drive a boat of light tonnage into shelter.

The majority of dredges of this class are provided with a suction pipe leading from each side of the vessel, the wave-motion being taken up by a joint of rubber pipe, 10 to 20 feet in length, while the weight of the suction pipe itself is supported by blocks and chains. In the latest types the machinery is placed amidships, with the hoppers in front and behind. The dredges for the Mersey, and two of those first used at New York Harbor (which were generally modeled after the Mersey dredge), were provided with a single suction pipe leading down a well in the center of the hull, but owing to the confined position the pipe was occasionally broken by the rolling of the ship, causing delay and expense in consequence.

Fig. 50.—Plan and Sectional Elevation of Dredge.

Fig. 51.—Cross-section of Pipe Line.

Fig. 52.—Cross-section of Dredge.

Fig. 51a.—Joints of Pipe Line.

DETAILS OF CONSTRUCTION, ETC., OF THE DREDGE "J. ISRAEL TARTE."

In working, the dredge steams slowly over the bar, the shoe of the suction pipe dragging along the bottom and leaving a furrow a few inches deep. The material is pumped into the bins in the ship, and when these are full, she proceeds to the dumping ground and deposits the load.

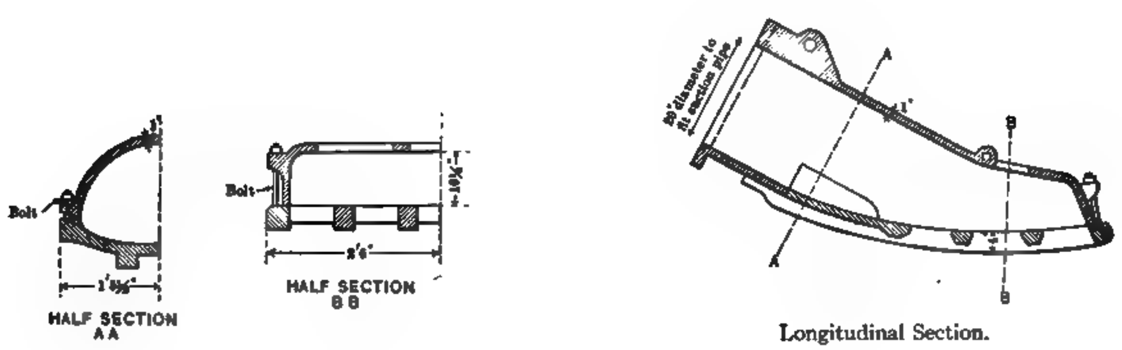
**Mississippi River Dredges.**—Owing to the shifting nature of the bed of the lower Mississippi, and to the consequent great cost and probable unsatisfactoriness of regulation works, a project was adopted in 1896 for maintaining by means of dredging, a low-water channel in this portion of the river, namely, between Cairo and the mouth, a distance of 1063 miles. (See also p. 88 and after, and p. 112.) The least depth was to be 9 feet, and the least width 250 feet. The method was finally adopted only after a considerable amount of experimental work had been done with hydraulic dredges of large capacity, which were able to open a channel through a bar within a short time sufficiently wide and deep to accommodate navigation. The commission having the matter in charge reported that the plan had met such success as to justify the continuance of dredging. They stated that it had proved to be "a successful, economical, and reliable means of low-water channel improvement," and subsequent experience has confirmed this belief. The work may require repetition at the same points every season, and sometimes as many as four times or more during the same season, but experience has shown that the material filled in is looser than the original river-bed, and that in the succeeding season the old channel can for that reason be reopened with less work than at first. From five to eight dredges are usually employed or kept in commission during the season; all are of the jet type (see Fig. 44*a*), those of more recent construction, the "Iota," "Kappa," and "Henry Flad," being provided with side paddle wheels 22 feet in diameter, for self-propulsion.* This has been found a marked advantage, as the towboats and tenders can be dispensed with, reducing the number of men employed from 60 to 45, and the dredges can be held in position better when at work, by using one or both of the wheels. Their hulls are of steel, 192 feet long, 44 feet wide and 7 feet deep, with a draft of about 48 inches, and with two 24-inch suction pipes. The suction shoes are $23\frac{1}{2}$ feet wide. The boilers are seven in number, of the Mississippi type, set in three batteries, and with a working pressure of 170 pounds. The main pump is centrifugal, with a 32-inch discharge, capable of delivering not less than 1000 cubic yards of sand per hour through 1000 feet of pipe, and is operated by a pair of horizontal tandem-compound condensing engines of 16-inch and 26-inch cylinders and 20-inch stroke, direct connected. The sand agitator is of the water-jet type, acting under a pressure from a duplex pump of 35 to 70 pounds per square inch. The pipe line is of $\frac{1}{4}$-inch steel plate, in 50-ft. sections, supported on pontoons and with swivel joints. (Fig. 57.) Twenty anchor piles, 35 feet long and 11 inches outside diameter, and of metal from $\frac{5}{8}$ inch to $\frac{3}{4}$-inch thick, are supplied with each dredge. Full quarters are provided, including laundry, bath-rooms, machine shop, and refrigerating and electric-light plants.

* See p. 133 for general dimensions, etc., of two of the older dredges.

Fig. 53.

FIG. 54.—U. S. Sea-going Dredge "Cumberland."

FIG. 55.—U. S. Sea-going Dredge "Burton."




Longitudinal Section.

FIG. 56.—Drag Shoe of U. S. Sea-going Suction Dredges, New York Harbor Entrance.

All the dredges are supplied with water-jet agitators to loosen the sand. Cutters and similar devices were tried, but owing to the then general lack of experience in their design, and to the comparative simplicity of the jet, as well as the trouble from sunken drift and snags, their use was abandoned. As the service of these dredges is very intermittent, and as they have to deal only with river-silt, it is probable that this simplicity amply compensates for the reduced efficiency as compared with the use of cutters. A somewhat lighter draft, which is a matter of importance, can also be employed.

*Location of Channel.*—In this class of dredging much depends upon the skill with which the location of the channel is made, as well as upon the stages of the water. No fixed rules can be laid down for the axis, as it is largely a matter of judgment, and each case must be studied specially. The worst bars are naturally found where the river is passing from a bend on one side to a bend on the other, called in America a " crossing." In such places, as all pilots know, the water loses more or less of its force and direction, and consequently of its stability of channel, yet seems always endeavoring to find the

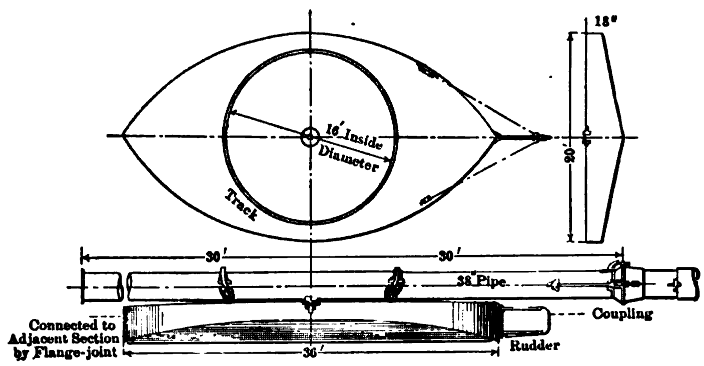



Fig. 57.—Pontoons for Supporting Discharge Pipes, Dredges "Kappa" and "Henry Flad." (Mississippi River.)

most direct way from deep water to deep water. It is usually along this path, which the survey shows in general outline, that the best results can be obtained. It should be noted, however, that experience has shown that the cut must be made along the line of the current. A cut improperly located, or located at a noticeable angle with the current is never maintained by the river, while one properly located is usually kept open, and often enlarged, during the low-water season.

The plan pursued in recent years has been to survey rapidly, some time before low water, the places most liable to require dredging. A second survey is made when it is desired to begin operations. A comparison is then made of the surveys, followed by further obstervations, and a decision is reached as to where the cut should be made. Sometimes, however, it becomes necessary to abandon a channel after it has been started, because conditions prove unfavorable to its maintenance, and at many locations it is necessary to remove additional material from time to time. During the season of low water patrol boats, carrying the survey parties, keep a constant watch upon the channels through the bars, and any shoaling is at once reported.

*Operation.*—To operate one of these dredges two wrought-iron anchor piles are sunk by a water-jet about 75 feet apart, and about 1000 feet above the point where the dredging is to begin. Wire cables are run to them from the dredge, which then commences pumping, slowly winding in the cables by steam-drums, the rate being of course commensurate with the capacity of the pumps, an average being between 60 feet and 80 feet per hour. In windy weather it is necessary to set side piles to steady the boat, and they are frequently required also as a safeguard against the cross-currents. After one cut is finished the piles are moved over for another cut, the dredge is dropped downstream, and the operations are recommenced. A ridge 8 to 10 feet wide is left between one cut and the next, as it has been found that such ridges are leveled down and washed away by the current. Thus a 250-foot width of channel takes 6 to 9 cuts, according to the width of the suction head. The excavated material is deposited through the discharge pipe in the deep water below the bar, at a considerable distance from the dredge. As the size of the river is so great compared with the volume of material dredged, this depositing of material has not been found to give further trouble below.

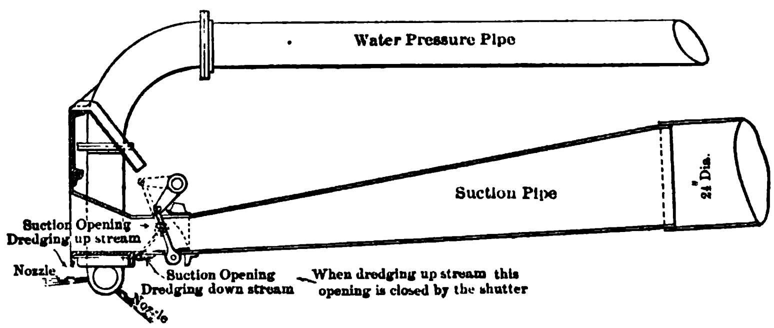



FIG. 58.—Suctionhead of Mississippi River Dredge "Delta." Designed for working up- and downstream.

The piles consist of hollow wrought-iron tubes, closed at the top and open at the bottom, with an attachment near the upper end for a $2\frac{1}{2}$-inch water hose. They are sunk from 15 feet to 20 feet into the sand, the mooring lines being attached to shackles near the river-bed. In the case of dredges which are not self-propelling, a special boat carries the apparatus used for sinking them; the others place the piles themselves. Mushroom anchors, similarly sunk by water-jet, are also used. Some of the dredges are fitted with special suction heads which allow of working both up- and downstream, saving a considerable amount of time when making a series of cuts. (Fig. 58.) A cut made by working downstream has also been found more effective, since the channel is opened more quickly by the flow of water. The width of suction head varies from 34 feet for the "Delta" to 24 feet for the latest types.

As an example of their rapidity of operation, a case is quoted of a bar with a channel-depth of 7 feet and a least depth in the proposed cut of $5\frac{1}{2}$ feet. In sixty-eight hours after its arrival the dredge had made six cuts of an average depth of 8 feet and a total length of 4440 feet, resulting in a channel of least dimensions of 200 feet wide and 12 feet deep. The time was distributed as follows: Placing plant, $4\frac{1}{2}$ hours; changing cuts, $3\frac{1}{2}$ hours; repairs, 1 hour; dredging, 59 hours. Total 68 hours.

During low water, if no rapid rise or fall occurs, the dredged channels are usually permanent, but if there is a rapid change of stage, or if snags or logs become caught near the head of the channel, a bar is liable to form very rapidly. Sunken logs, etc., form one of the principal difficulties met with in this dredging. The cut is always made parallel with the current, though this may not give the shortest distance across the bar. Experience has shown that it is practically impossible to maintain a channel at a considerable angle to the line of flow. Moreover, where the line of the current is followed, a large amount of scouring is done by the river. Work is begun on a bar as soon as the depth of water will permit, so that advantage may be taken, with this end in view, of the volume of the flow.

The working season is during the period of low water, and usually extends from the middle of August to the middle of December. During this time part or all of the dredges are held in commission, and when at work on a bar operations are continued night and day. The greater portion of the work is comprised in some 250 miles of river, although the portion which may require dredging extends for about 500 miles below Cairo. Pls. 7, 8, and 9 illustrate the character of the work.

Local experiments were made in 1907, in connection with a project to obtain a waterway of 14 feet depth between Chicago and the Gulf of Mexico, to determine whether the adopted channel depth of 9 feet through the bars could be increased and maintained at 14 feet. The experiments proved entirely successful, but it appeared that the cost for a 14-ft. channel below Cairo would amount to about $3100 per mile per year, as compared with an average of $250 for the 9-ft. channel.

The following tables give information as to cost, distribution of time, etc.

| Season Ending in Year. | Number of Dredges in Commission. | Total Time in Commission, all Dredges. Hours. | Time Towing and Making up Tow. Per Cent of Total. | Time in Repairs, Per Cent of Total. | Time in Actual Dredging Operations. Per Cent of Total. | Time in Waiting Lower Stage. Per Cent of Total. | Number of Bars Dredged. | Number Dredged once only. | Number Dredged Twice. | Number Dredged Thrice. | Number Dredged Four Times. | Total Length of Cuts, Feet. | Minimum and Maximum Average Advance per Hour, Feet. | Minimum and Maximum Average Depth of Cut, Feet. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1898 | 5 | 8,855 | 7.1 | 7.5 | 37.1 | 48.3 | 13 | 10 | 1 | 1 | 1 | 183,990 | 21.0 to 206.0 | 1.7 to 6.7 |
| 1899 | 5 | 12,144 | 10.1 | 10.6 | 31.7 | 47.6 | 11 | 6 | 1 | 2 | 2 | 329,925 | 40.0 " 188.0 | 2.0 " 6.3 |
| 1900 | 6 | 10,368 | 5.5 | 9.3 | 18.2 | 67.0 | 12 | 9 | 2 | ... | ... | 201,695 | 37.9 " 251.5 | 2.1 " 5.6 |
| 1901 | 6 | 15,240 | 4.7 | 5.3 | 23.5 | 66.5 | 18 | 11 | 1 | 1 | 5 | 496,235 | 47.4 " 398.0 | 1.9 " 9.8 |
| 1902 | 7 | 14,521 | 6.9 | 3.8 | 13.5 | 75.8 | 11 | 8 | 4 | ... | ... | 202,455 | 42.4 " 198.5 | 2.2 " 5.6 |
| 1903 | 6 | 13,159 | 10.2 | 2.3 | 19.5 | 68.0 | 15 | 6 | 4 | 3 | 1* | 232,610 | 17.0 " 451.0 | 0.9 " 7.6 |
| 1907 | 5 | 9,624 | 4.4 | 5.0 | 16.4 | 74.2 | 12 | 3 | 4 | 3 | 2 | 163,395 | 92.0 " 125.0 | 4.4 " 6.4 |
| 1910 | 7 | 21,288 | 7.5 | 4.2 | 18.4 | 69.9 | 23 | 14 | 7 | 2 | ... | 532,755 | 121.0 to 252.0 | 3.8 " 5.5 |

"Actual dredging" includes placing plant, changing cuts, etc. Ordinarily one dredge cannot open more than two bars before the river falls below the 9-ft. stage.

The following were the total cubic yards dredged for the years given: 1899, 1,612,000; 1900, 1,146,000; 1901, 1,667,000; 1902, 813,000; 1903, 891,000; 1907, 1,152,000; 1910, 2,471,000. The largest quantity between 1899 and 1910 was removed in the latter year, and the smallest quantity was removed in 1905 and amounted to 198,000 cubic yards.

* One bar was dredged five times.

The following were the expenses of the dredges, tenders, survey parties, etc., during the several 12-month periods comprising several working seasons from 1902 to 1910.*

| Season Ending During Year. | No. of Dredges in Commission. | Total Expenses for the 12 Months. $ | Percentages of Total Expenses. | | | | Expenses While in Commission. $ | Percentages of Same. | | | | Approximate Cost per Mile of River. (1063 miles). |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Care of Plant When Laid Up. | Annual Repairs. | New Machinery and Sundries. | Expenses While in Commission. | | Labor. | Board. | Fuel. | Repairs and Sundries. | |
| 1902 | 6 | 237,500.00 | 16.4 | 15.7 | 13.3 | 54.6 | 129,700.00 | 54.8 | 11.9 | 26.9 | 6.4 | $222.00 |
| 1903 | 7 | 208,300.00 | 22.2 | 23.2 | 6.6 | 48.0 | 99,600.00 | 56.4 | 13.3 | 26.0 | 4.3 | 196.00 |
| 1907 | 5 | 229,800.00 | 23.6 | 21.8 | 11.1 | 43.5 | 100,200.00 | 53.7 | 11.7 | 29.4 | 5.2 | 216.00 |
| 1910 | 7 | 392,100.00 | 14.9 | 18.0 | 27.4* | 39.7 | 155,200.00 | 53.9 | 16.7 | 27.0 | 2.4 | 368.00 |

* Dry-dock, $93,140, included.

**Dredging on the Volga.**—The river Volga, which is similar to the Mississippi in size and discharge, though of less slope and consequently of more stable bed, affords another instance where dredging has proved a more certain and economical means of improvement for a large river than dikes or similar expedients. Some attempt was made to employ dikes, but owing to their great lengths and cost dredging was substituted, and is now depended on to maintain navigation over more than 2500 miles of the Volga system. The period of navigation lasts from six to eight months only, owing to the severe winters, but during that time an immense traffic is carried on, the bulk of which travels over that portion of the river between Rybinsk and the mouth, a distance of 1653 miles. (See p. 85.) The low-water discharge varies from 3900 to 113,000 cubic feet per second, and the maximum from 223,000 to 1,426,000 cubic feet per second, according to the locality. The width in low water varies from 800 feet in the upper sections to a maximum of 9000 feet near the mouth. High water rises about 12 feet at the head of the Delta, and about 50 feet at the mouth of the Kama, and the total fall in the 1653 miles is 319 feet, an average of only 0.19 foot per mile. There are some 370 bars and other obstructions in this length, with depths which have varied in the last twenty years from 28 to 126 inches, and it is on these bars that the dredges are employed, usually in July, August and September. The least dimensions of channel desired are from 250 to 260 feet in width and from $4\frac{1}{2}$ to 7 feet in depth, according to the locality, the increase of low-water depth obtained by dredging varying from $2\frac{1}{2}$ to $3\frac{1}{4}$ feet. The cuts do not fill nor change to the extent met with on the Mississippi, although the work has sometimes to be repeated two or three times in one season on the same bar. This, however, is rare, and in some sections less than half the bars need annual dredging. During the season of 1901 the hours of effective work of the fleet on the Volga varied from 22.6 to 80.2 per cent of the total hours in commission. The average of this total spent on dredging bars was 56 per cent; on deepening winter harbors, 22 per cent; and on towing, repairs, etc., 22 per cent.†

* See also p. 122. The monthly cost of operation in the field is usually taken as $5500 for non-self-propelling dredges and as $4500 for self-propelling dredges. For additional information as to cost, etc., see "Engineering Record," March 21, 1908, or the Annual Report, Chief of Engineers, U. S. A., for 1910.

† Transactions Am. Soc. C.E., vol. lii (S. P. Maximoff), and Reports of International Navigation Congress, 1912.

In 1909 there were 24 dredges in the Volga fleet, with a total nominal capacity of 8700 cubic yards per hour, and the total amount dredged from the main river and its tributaries, the Oka and the Kama, was 6,300,000 cubic yards. The cost averaged $357 per mile.

The number of dredges on Russian waterways in 1901 was 70, chiefly of the elevator or ladder type, with a total capacity of 13,700 cubic yards per hour, and an invested capital of about $3,530,000. The cost of purchase, maintenance, and operation rose from 11 per cent of the total appropriations for river and harbor improvements in 1888 to 39 per cent in 1903. The cost of maintenance and operation in 1901 was about $908,900, of which $403,290 were spent on the Volga. The depths in which work was carried on varied from $1\frac{1}{2}$ to 12 feet, and the number of hours of effective work from 22.6 to 80.2 per cent of the total number of hours in the season. The total length of cuts made aggregated $145\frac{1}{2}$ miles, from which 9,260,000 cubic yards were removed at an average cost of 9.8 cents per cubic yard, the minimum being 2.7 cents and the maximum 48.4 cents. One cutter dredge of modern type is stated to have removed 2,980,000 cubic yards in 1902 at a cost of 3.6 cents. This dredge was provided with eight cutters, covering the entire width of the boat.

The following table gives details as to cost, etc.:

| Year. | 1899 | 1900 | 1901 |
|---|---|---|---|
| Total length of cuts, miles | 56.9 | 80.1 | 145.5 |
| Cubic yards excavated | 5,160,300 | 6,000,900 | 9,261,400 |
| Total cost | $516,500 | $590,700 | $908,900 |
| Average cost per cubic yard in cents | 10.0 | 9.8 | 9.8 |

Dredging is also used to maintain open navigation on several other rivers in Russia, and in other European countries, as well as on the Uruguay, the Paranà, certain waterways in India, and elsewhere.

**Efficiency and Tests of Hydraulic Dredges.**—The efficiency of a hydraulic dredge, or the ratio of the amount of excavation to the amount of water pumped, varies with the type. Tests made with a sea-going dredge on the Atlantic coast, where the sand was loosened by the shoe of the suction pipe being dragged along the bottom, gave a maximum efficiency of 18 per cent. Tests with two of the large Mississippi dredges, which use jets to loosen the material, gave an efficiency from 12 to 28 per cent, the mean being 19 per cent. The proportion reached the maximum when working against the current, as would be expected.* The efficiency under ordinary working conditions in the material usually met with has been stated to be about 15 per cent.

The efficiency of the cutter type is considerably more than the foregoing, but there are few tests on record. In one case a dredge with a 20-inch suction pipe removed 28,990 cubic yards (from measurements in place) of clay in an actual working time of twenty

* Annual Report, Chief of Engineers, U. S. Army, 1903.

hours and forty-five minutes, and the net result showed the efficiency to be close to 40 per cent. In other tests made with larger dredges, working in sand and mud, the maximum reached was 35 per cent.

In 1903 special tests were made on seven of the Mississippi dredges, in order to determine their capacity, etc.* These investigations indicated that the form of suction head had little effect on the efficiency, and that it was of much greater importance to design it to withstand severe usage than to be theoretically perfect. It was found that for the maximum efficiency the area of its openings, and the area of both suction and discharge pipes, should be approximately the same, thus preserving an equal velocity. The loss from friction in the pipes was very largely increased where short bends were employed. The most effective speed for pumps of the size used (24-inch to 30-inch suction) was found to be about 50 feet per second for the peripheral velocity, producing a head of from 40 to 50 feet. The jet method of disturbing the material was found to be practically useless except with sand or silt.

The following special tests show the approximate maximum capacity of hydraulic dredges, according to their horse-power. The first two were made in 1898 with Mississippi dredges running a net time of 25 to 27½ hours, with 141 to 178 average revolutions per minute of the main pumps. The last is a test of the cutter dredge described on page 103, and built in 1901 for use on the St. Lawrence River.†

| Dredge. | Cubic Yards Moved, Total. | Cubic Yards per Hour. | Distance Deposited Feet. | Diameter of Ditch-Pipe, Inches. | Indicated Horse-power. | Cubic Yards per Horse-power per Hour. | Depth of Dredging, Feet. | Material. | Cost per Cubic Yard. |
|---|---|---|---|---|---|---|---|---|---|
| Delta......... | 34,460 | 1259 | 1000 | 34 | 1050 | 1.19 | 16 to 18 | Light sand.... | 0.7 |
| Epsilon....... | 32,410 | 1306 | 1000 | 32 | 748 | 1.74 | 16 to 18 | Light sand.... | 0.9 |
| J. Israel Tarte.. | ....... | ...... | 2000 | 36 | 980 | 2.15 | 35 | Soft blue clay.. | ... |

The "Delta" is said to have dredged for a short time at the rate of 2550 cubic yards per hour.

**Silting-up of Dredged Channels.**—The rapidity with which a dredged channel will silt up depends on its location with respect to the direction of the current to which it may be exposed, on the amount of sediment carried in the water, and on the material of the bed. It is in fact important, as in the training of a river, to retain as far as possible the natural conditions of flow, or the work of maintenance may be unduly burdensome. If, for instance, a cut is located at an angle to the current, the edges of the upstream side will begin to be washed in, and gradually a large amount of matter will come into the channel, and by its effect on the incoming flow will create additional complications. The cut in such cases also forms a basin where the current is more or less checked, and where in consequence it will deposit sediment. If, on the other hand, the dredging coincides with the natural current, the velocity of the latter may be increased rather than checked,

* Annual Report, Chief of Engineers, U. S. Army, 1903.
† Annual Report, Chief of Engineers, U. S. Army, 1898, and Transactions Can. Soc. C. E.

and it will carry its sediment along, and by flowing parallel to the sides of the cut, will have little tendency to erode them. On shallow bars the water, in seeking the line of least resistance, tends to concentrate in the deeper artificial channel, and if the latter is judiciously located, much scouring may be done by the water itself. This the experience with the Mississippi dredges has proved, for where the work follows the flow, the river will usually keep the cut open until the next flood, while the greater the angle of inclination is made the faster will the cut fill up. On one occasion one of the dredges, making a cut upstream partly across the current, found that she was rapidly becoming embedded in sand, and had to abandon the work and make vigorous efforts to regain deep water. On the bar at Charleston Harbor, some miles out in the Atlantic, a similar tendency has been observed for the water to assist the dredging by its scour, and in the improvement of the Southwest Pass at the mouth of the Mississippi the tendency was very marked. Under the most favorable conditions, however, it may be taken for granted that if dredging is carried out in an alluvial river without additional artificial contraction of flow, it will sooner or later have to be repeated.

There is an intermediate class of rivers, however—those of moderate regimen—where the work remains more or less permanent. An example of such is the Upper Fox River in Wisconsin, whose bed consists principally of sand, but which carries little sediment, and whose floods have a range of only a few feet. The dredging in it covered many miles, and ranged to 6 feet in depth, but in spite of the sandy material, no great tendency was found, during a period of several years, for the cuts to fill up. Similarly the River Severn in England was dredged at various points in a distance of 42 miles in order to procure a low-water depth of 10 feet for the first 30 miles, and of 12 feet for the remaining 12 miles. As is the case with the Fox, the Severn is also canalized, but it carries a considerably greater proportion of sediment. The cuts were made in various materials from marl to gravel and fine sand, and an experience of ten years showed little tendency towards silting up. The only cases where such occurred was in the slack water of the lock entrances and at certain points where the dredge-cuts had to be located on convex shores, and the latter trouble was overcome by the use of training walls on the concave side for the purpose of confining the water to the artificial channel.

On the Seine below Paris, canalized by movable dams for a channel depth of 10½ feet, bars and shoals originally existing were found to be reproduced to a greater or less extent after each flood, and had to be dredged out. Certain of these places were later regulated by dikes, as would have been done with an open river. The channel dredging between 1900 and 1910 amounted to 500,000 cubic yards, and the dredging at the locks and dams to 755,000 cubic yards.

The best class of rivers is the one where the banks and bed are of a more or less stable nature, as gravel or clay, in which the sides of the cuts act like training walls in preventing deposit. To this class belongs the St. Lawrence, and to a less degree parts of the Ohio. In the former, dredging made sixty years ago has remained almost unchanged,

and in the work of later years, where the cuts run to 20 feet in depth, there has been little or no filling in. The material is largely of blue clay, intermixed with stones. On the upper Ohio the dredging consisted chiefly of bettering the width and depth of the low water channels through the gravel bars, the maximum depth of cut being only a few feet. Much of the work remained clearly defined after a lapse of twenty-five years, and practically the only places where re-dredging was found necessary was in the bars built up by creeks at their outlets, or where deposits were caused by sliding banks. Where possible the excavated material was placed to act as a training wall, or dikes were built to increase the flow through the dredge cut. (See also p. 80.)

When, however, the bed of a river consists of a stable material and this material is removed by dredging so as to expose less compact strata below, there will usually occur more or less erosion, and this will continue until the river has worked out a cross-section suitable to the quality of the material. Thus in dredging in the Hudson River in the neighborhood of Troy, where a channel had to be excavated up to a depth of 8 feet in material composed of sand and gravel, the original bed was covered with a closely-packed layer of large and small gravel. When this was removed it exposed the looser sand and gravel below, and for some years there was found to be a considerable amount of sand washing into the new channel and evidently coming from the surface of the newly-exposed strata. After a while, however, when the river had washed away the loose sand, it was found that the bed had again become covered with a layer of gravel and that there was no further erosion.

In the improvement of harbors, dredging has sometimes to be done across a wide, shallow bay, exposed for some miles on each side to the winds and waves. Such cuts invariably tend to silt up; the changeable action of the natural forces usually forms no well-defined channel or direction of flow, and the artificial channel acts as a catch-basin for the traveling silt. Examples of these conditions are to be found in the bays of Mobile and Galveston, along each of which dredge-cuts had to be made many miles in length. In the latter the material was deposited in the water several hundred feet from the side of the new channel, but the dredged depth in two years filled up at some places with 10 to 12 feet of silt. Much of this apparently came from the spoil banks, and the amount would accordingly be reduced in course of time, but it has been found that such cuts in similar locations have shown a persistent tendency to fill. Where they are much used, the disturbance of vessels passing, and especially that of the propellers, increases this tendency considerably by washing in the sides.

**Dredges for Canalized Rivers.**—The type of dredge for a river possessing locks and dams, where these are sufficient in importance to require annual service of this nature, will generally be determined by the character of material to be handled and the method of its disposal, but the dredge of most general application is the dipper dredge, fitted for use with a grapple bucket also. Its capacity should be not less than 2 yards, as small dredges are very uneconomical, costing almost as much in operation as large

ones. The machinery should be of ample power, as part of the work will consist in tearing out wrecks, snags, old cribs, etc., and the strength of all parts should be designed accordingly. The limiting depth of water in which to work should be not less than 18 or 20 feet, as the dredge will then be practically independent of summer rises, and can work on uninterruptedly. This is sometimes a matter of much importance, as we have more than once seen important works delayed in times of pressure because a rise of a few feet put the dredges out of operation where their limit of depth was only 12 to 15 feet. The coal capacity should be ample, so that the work can be carried on for several weeks away from the base of supplies.

Where the material is sand and gravel and where the works are of a permanent nature, without cribbing, etc., the hydraulic dredge is more satisfactory and economical in operation than the dipper dredge, as the disposal of the material can frequently be accomplished without the assistance of a towboat or of scows.

Steel-hulled dredges and scows have come into use in recent years, and have given excellent satisfaction. They are much stiffer than the wooden hulls, and give no trouble from leakage, and if they are properly cared for appear to outlast two or three of the wooden type.

Scows for river work should preferably be of the side-dump variety, so they can be used for backing dams, filling washouts, etc. The bottom-dump scow is usually inapplicable for such purposes, as it needs a depth of water of 8 to 10 feet for dumping.

**Channel Deepening by Dynamite.**—The use of dynamite for assisting the removal of sand-bars has often been proposed, but seldom applied, and then with uncertain measure of success. It was used on the Brunswick Bar in Georgia from 1891 to 1895, and also at Aransas Pass, Texas. At the former locality about 100,000 pounds were used, with the results as shown by Government surveys that the four years' work had only deepened the channel 1.7 feet. Dredging was then resorted to, and a further depth gained by this means of 1.6 feet in eight months, or almost as much as in the whole four years preceding.

The advocates of the method of dynamite, among whom have been experienced engineers, claim that the explosion loosens the material for a wide distance, and thus enables the current to remove it. Experiments with eggs buried in the sand showed that one was broken at a distance of 110 feet, while others, from 115 feet to 315 feet away, were unharmed, the charges being, it is reported, about 100 pounds.*

* "Engineering News."

## COST OF DREDGING *

**Sea-Going Suction Dredges.** *River Mersey.*—Two large hopper dredges were employed on the bar, one of which, the G. B. Crow, built in 1895, had a carrying capacity of 2000 cubic yards. She had two pumps with one suction pipe 45 inches in diameter, leading down through a central well, and could dredge to a depth of 54 feet. The maximum output of the other dredge, which cost $325,000, and was built in 1893, was given as 26,000 cubic yards of sand in twenty-four hours, or 1083 yards per hour. It is stated that in 1898 2,900,000 cubic yards were removed by the Crow at a cost, including dumping, of 1.22 cents per yard. Of this amount wages were 0.46 cent, supplies .005 cent, and repairs 0.26 cent. This was the cost of operation only. A suction dredge of similar type was completed for this bar about 1910, with an estimated capacity of 50,000 cubic yards in twelve hours. The channel depth desired was from 35 to 40 feet.

*New York Harbor.*—The contract price in force in 1904 for dredging $42\frac{1}{2}$ millions of cubic yards of sand and gravel in opening the Ambrose Channel from New York Bay to the Atlantic, in order to provide a channel 2000 feet wide and 40 feet deep at mean low tide, was 9 cents per yard. This work was done by two twin-screw drag-suction dredges. During one period of 562 working days, work was done on 442 days, 88 days were lost in repairs, and 32 because of bad weather. During the 442 days the average working time was $15\frac{1}{4}$ hours per day, the lost $8\frac{3}{4}$ hours being due to coaling, minor repairs, stress of weather, etc. The operating cost was about $4500 per dredge per month, but about 16 per cent of the work was done below the required level, as the dredges made very uneven cuts.

The cost of removing 111,600 cubic yards of sand and gravel by the United States' suction dredge Gedney from the foregoing channel during seven months in 1902-3 averaged 14.7 cents per yard. For the first four months, December to March, the cost was 20.5 cents, owing to rough weather, etc., while for April, May and June, it was 10.7 cents per yard. During a period of 365 days, she worked 109 days; 114 days were lost from fog, storms, etc.; and the remainder was spent in repairs, Sundays, etc.

The cost of removing $32\frac{1}{2}$ million cubic yards of sand, mud, and gravel from the sea channel into New York Bay prior to 1898 was 10 cents per yard, from a maximum depth of 35 feet below low water, the work being done by the Government. Two dredges specially constructed for this work removed during the eleven months subsequent to July 1, 1905, about $3\frac{1}{4}$ million yards at a cost of 5.27 cents per yard (field expenses

* Data compiled from Annual Reports, Chief of Engineers, U. S. Army, Reports of International Congresses of Navigation, etc. The costs of Government work do not include interest or depreciation, or the sundry other expenses for which a contractor must allow. The reports from which the figures are taken do not always state whether the prices are for contract or hired-labor dredging. Detailed tables of sizes, costs, work done, etc., for United States dredges will be found in recent Annual Reports, Chief of Engineers, U. S. A. The dimensions and cost of some typical dredges are given at the end of this chapter. See also "Engineering News," May 9, 1912.

only). Of this amount labor cost about $33\frac{1}{3}$ per cent; subsistence 9 per cent; repairs, etc., $25\frac{2}{3}$ per cent; coal, 27 per cent; sundries, 5 per cent. The time occupied in going and returning from the dumping grounds was about one-third of the total time.

*Atlantic Coast and Gulf of Mexico.*—In 1903 a Government drag-suction hopper dredge removed 386,500 cubic yards of fine sand at a total cost of 5.6 cents per yard, from a depth of 20 to 30 feet. Effective time was 55 per cent of the total time, and time lost from bad weather, repairs, holidays, etc., was 45 per cent.

In the same year a similar dredge removed 330,000 cubic yards of coarse sand from outlets along the middle Atlantic coast at an inclusive cost of 7.3 cents per yard. The U. S. dredge "Charleston," removing sand and mud from certain points on the Gulf of Mexico, dredged between July 1, 1909, and June 30, 1910, about 750,000 cubic yards at a net cost for work only of $3\frac{1}{2}$ cents per yard, and of $5\frac{1}{2}$ cents including all expenses. The average distance to the dump was half a mile; the average dredging was about 3000 cubic yards per day, and per hour about 650 yards. The average depths of cut varied from 3 to 12 feet, and the finished depths below tide from 22 to 30 feet. The total running cost for the twelve months was about $40,000.

The U. S. dredge "Delaware" during the same period removed 3,850,000 cubic yards of sand, mud, clay, and gravel from the Delaware River at a net cost for work only of $2\frac{1}{10}$ cents per yard, and of $2\frac{1}{2}$ cents including all expenses. The average distance to the dumps was 5.7 miles. The average daily dredging was 14,800 cubic yards, and per hour about 1130 yards. The average depth of cut was about 5 feet, and the finished depth below tide from 30 to 35 feet. The total running cost for the twelve months was about $96,000.

The maximum cost per yard for this class of dredge during the period named was from 10 to 16 cents, not including extraordinary repairs.

*Kaiser Wilhelm Canal.*—At the Elbe entrance to this canal the river deposits annually some 640,000 cubic yards of silt, which is removed by a drag-suction hopper dredge. The material is taken about two miles to the place of deposit. About 4600 cubic yards are removed in ten hours at an average cost for working expenses of 8 cents per yard.

*Lower Weser.*—Between 1888 and 1898, 50,358,000 cubic yards of material, chiefly clayey sand, were removed in part by contract and in part by hired labor. The total cost where the work was done by a sea-going hopper dredge was about 2.2 cents per yard, and where done by elevator dredges, about 2.4 cents per yard. Where suction dredges of the ordinary type were used, the cost, including a rehandling and discharging, was 7 cents per yard.

**Suction Dredges in Rivers and Harbors.** *In France.*—The average inclusive cost of removing sand from the Adour, Loire, and Seine, in 1898 was from 1.8 to 4.5 cents per cubic yard, from maximum depths of $29\frac{1}{2}$ to 40 feet. One dredge on the Adour was provided with special pipe joints, permitting work in waves from 6 to 7 feet high. These dredges were of the suction type without cutters.

*Ostend Harbor.*—The contract price for removing about a million cubic yards of sand by suction dredging between 1890 and 1896 was from $7\frac{1}{3}$ to 11 cents per cubic yard. The material had to be carried $2\frac{1}{2}$ miles to the place of deposit, and owing to exposure dredging was suspended about one-half of the time. The amount annually dredged to maintain a depth of 20 feet below low water was stated in 1904 to be about $1\frac{1}{3}$ million cubic yards, costing by contract from $4\frac{1}{2}$ to 7 cents per yard.

*In the United States.*—The contract price for making a canal for the diversion of the Yazoo River, at Vicksburg, Mississippi, between 1900 and 1903, involving the excavation of $7\frac{1}{2}$ million cubic yards, was 12.4 cents per yard. The material was blue clay and sand, and was deposited chiefly on the land near by. Two 20-inch hydraulic cutter dredges, of actual capacities of 260 and 290 yards per hour, and one clam-shell dredge, were used. The actual cost was said to have been considerably below the contract price.

Special tests (see p. 116) on the Government dredges which maintain the channel in the Mississippi River below Cairo, gave the net cost of dredging only as from 0.7 to 0.9 cent per cubic yard. These were suction dredges with water-jet agitators. It is not practicable to give a general working cost per cubic yard for this work, as in many cases the current removes as much or more than the dredge, so that close measurement is impossible. In addition, the dredges may be kept fully occupied, or may do very little work during a season, depending on the stages of water. It may be said, however, that it costs in round numbers about $250,000 a year, or about $250 per mile, to ensure the existence of a 9-foot channel from Cairo to the Gulf, or $1000 per mile for that portion of the river which needs most work. The items of cost are given in detail on p. 114.

The contract price for dredging about 2,100,000 cubic yards in Galveston Harbor, Texas, in 1903 was from 8.3 to 8.8 cents. The material was about one-quarter light sand and three-quarters tough clay. Cutter dredges were used, and the material was deposited through pipes at an average distance of 850 feet. The maximum depth dredged was 32 feet below low-water.

The contract price for dredging 8,000,000 cubic yards in widening and deepening a waterway to the city of Houston, Texas, in 1903, was 14 cents. The material was chiefly tough clay, much of which was excavated from the banks, and was deposited through pipes at distances from 200 to 500 feet. The maximum depth dredged was 19 feet below low water. The work was done by cutter dredges.

The U. S. dredge "Barnard," with a 36-inch pump but no cutter, dredged during twelve months of 1909–1910 3,000,000 cubic yards of mud from Gulfport Harbor, Mississippi, at a gross cost of $3\frac{6}{10}$ cents per yard. The material was deposited close by, and was easily handled. The average daily output was 12,700 yards, and the hourly 1300 yards. The average depth of cut was 4.8 feet, and the finished depth below tide, 22 feet. The total running cost for the year was about $110,000.

The U. S. dredge "Cataract," with a 20-inch pump and a cutter, dredged during the same period 1,200,000 cubic yards of sand, stones, clay, etc., from the Delaware River at a gross cost of about 5 cents per yard. The average length of discharge was about 1500 feet. The average daily output was about 7000 cubic yards, and per hour about 380 yards. The average depth of cut was 14 feet, and the finished depth below tide, 32 feet. The running cost for the year was about $60,000.

For small dredges and limited work (under hired labor), the cost ran as high as 12 cents per cubic yard.

*Russian Waterways.*—The cost of this dredging is stated to be from 9.8 to 10 cents per yard. (See p. 115.)

**Elevator Dredges.** *Antwerp.*—The contract price for the excavation of loose stone and clay by elevator dredging, between 1896 and 1898, on the Scheldt below Antwerp was 4 cents per cubic yard for material transferred in scows for 1 mile and dumped in deep water, and 9 cents for material transferred 2.2 miles and deposited on shore.

*In France.*—On the Garonne, Charente, and Loire the average cost of removing by elevator dredges 1¼ million cubic yards of clay and gravel, between 1895 and 1898, was 7.2 cents per yard, including transferring in steam scows from a short distance up to 22 miles. The cost of transfer and dumping only for the 22 miles was 9.3 cents per yard, while the cost of dredging only, including repairs, office expenses, etc., was from 2.2 to 9.4 cents per yard. The work was done by hired labor.

*In Germany.*—In the Stettiner Haff the average cost of removing 15½ million cubic yards of sand and clayey mud, between 1893 and 1897, discharging principally by scows, was 4.7 cents per yard, while in the Kōnigsberg Canal, the inclusive cost of removing about twelve million cubic yards of sand and clay was 4.4 cents per yard, most of the material being placed upon the bank. In both cases elevator dredges were the chief means employed.

*In America. St. Lawrence River.*—In 1903 a total of 6,544,600 cubic yards, scow measurement, or 5,235,684 cubic yards, place measurement, of material, chiefly clay, was removed by Government labor from this river by six elevator and one suction cutter dredge at an average inclusive cost of 3.9 cents per cubic yard. The maximum depth of cut was about 20 feet, and of water, about 30 feet. The following table gives details of unit cost:*

* Report of Canadian Minister of Public Works. A summary of costs, etc., for 1911 is given in "Engineering News," November 7, 1912.

SUMMARY OF WORK DONE BY THE DEPARTMENT OF PUBLIC WORKS IN CANADA IN DREDGING THE RIVER ST. LAWRENCE SHIP CHANNEL BETWEEN MONTREAL AND QUEBEC IN THE FISCAL YEAR ENDING JUNE 30, 1903.

| Dredge. | Kind. | Dimensions, Feet. | | | | | When Built. | Working Capacity per Day, Cubic Yards. | Locality of Dredging.* | Time of Service, Days. | Nominal Working Time 20 Hours per Day. | Hours actual Dredging. | Number of Cubic Yards Dredged, Scow Measurement. | Total Cost of Operations of each Dredge and Plant. | Average Cost per Cubic Yard for each Dredge. Cents. | Kind of Material Dredged.† |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Over All Length. | Beam. | Hold Depth. | Average Draft. | Greatest Working Depth. | | | | | | | | | | |
| Lady Aberdeen | Elevator | 148 | 32 | 13 | 7.5 | 42.5 | 1900 | 4000<br>5000 | (1) | 192 | 3690 | 2736¼ | 982,750 | $32,287.23 | 3.28 | (8) |
| Lady Minto... | Elevator | 148 | 32 | 13 | 7.5 | 42.5 | 1900 | 2000<br>4000 | (2) | 190 | 3645 | 2615½ | 643,600 | 3c,652.48 | 4.76 | (9) |
| Laurier....... | Elevator | 168 | 32 | 14 | 11 | 42.5 | 1897 | 2000<br>3000 | (3) | 172 | 3305 | 1989½ | 365,630 | 31,861.19 | 8.71 | (10) |
| Laval......... | Elevator | 150 | 30 | 14 | 11 | 43.5 | 1894 | 1000<br>2000 | (4) | 155 | 2972 | 2050 | 273,420 | 37,342.62 | 13.65 | (11) |
| Lafontaine.... | Elevator | 168 | 32 | 14 | 9 | 45 | 1901 | 5000<br>6000 | (5) | 196 | 3780 | 2392½ | 1,162,500 | 33,503.71 | 2.88 | (12) |
| Baldwin....... | Elevator | 165 | 34 | 14 | 8 | 45 | 1902 | 2500<br>3500 | (6) | 191 | 3528 | 2670¼ | 783,500 | 32,282.81 | 4.12 | (13) |
| J. Israel Tarte.. | Hydraulic. | 160 | 42 | 12½ | 6 | 50 | 1902 | 20000 | (7) | ... | ... | ... | 2,333,200 | 57,846.51 | 2.48 | (14) |
| Totals........ | ........ | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | ... | 6,544,600 | $255,776.55 | 3.90 | |

* (1) Contre Cœur: Contre Cœur Bend, St. Ours Bend, Petit Traverse. (2) Contre Cœur: Petit Traverse, Contre Cœur Traverse. (3) Pointe Aux Trembles (en haut). (4) Pointe Aux Trembles, Pointe Citrouille, Champlain. (5) Contre Cœur: Contre Cœur Course, Contre Cœur Bend, Upper half Bellmouth Curve. (6) Petit Traverse, Pointe Aux Trembles (en haut). (7) Lake St. Peter, White bend No. 2 Curve, No. 2 Curve.

† (8) Blue clay, clay and stones, stiff blue clay. (9) Stiff blue clay, blue clay and stone. (10) Clay and stones. 11) Clay and stones, sand.—(12) Blue clay. (13) Clay and stones. (14) Soft blue clay.

CLASSIFICATION OF DISBURSEMENTS IN PERCENTAGES.

| Dredge. | *a* * | *b* | *c* | *d* | *e* | *f* | *g* | *h* | *j* | *k* |
|---|---|---|---|---|---|---|---|---|---|---|
| Lady Aberdeen.... | .13 | .19 | .08 | .04 | .16 | .05 | .01 | .26 | .08 | 1.00 |
| Lady Minto...... | .13 | .19 | .08 | .04 | .12 | .04 | .01 | .30 | .09 | 1.00 |
| Laurier.......... | .14 | .19 | .06 | .04 | .18 | .05 | .01 | .25 | .08 | 1.00 |
| Laval........... | .11 | .15 | .05 | .03 | .36 | .06 | .01 | .15 | .08 | 1.00 |
| Lafontaine....... | .17 | .19 | .07 | .05 | .14 | .05 | .01 | .24 | .08 | 1.00 |
| Baldwin......... | .16 | .18 | .08 | .03 | .17 | .05 | .01 | .24 | .08 | 1.00 |
| J. Israel Tarte.... | .23 | .15 | .05 | .03 | .15 | .05 | ...... | .25 | .09 | 1.00 |
| Totals........ | .16 | .17 | .06 | .04 | .18 | .05 | .01 | .24 | .09 | 1.00 |

* *a*. Fuel. *b*. Wages. *c*. Board. *d*. Stores and Materials. *e*. Repairs, labor. *f*. Proportion of general and office expenses, etc. *g*. Stone lifter service, elevator dredges. *h*. Tug service. *j*. Inspection, towing, sweeping, etc. *k*. Total cost of operations of each dredge and plant.

*Delaware River.*—The contract price for removing 10,900,000 cubic yards (scow measurement) of clay, gravel, and mud from the Delaware River in 1902 was 13.9 cents per cubic yard. The work was done by clam-shell, dipper, and hydraulic dredges.

**Dipper and Clam-Shell Dredging.**—The inclusive cost of moving by hired labor 428,200 cubic yards from rivers in North Carolina in 1903 by a clam-shell dredge was 8.0 cents per yard.

On the Fox River, Wisconsin, where both clam-shell and dipper dredges are employed by the Government, 163,000 cubic yards of sand and clay were removed in 1900 at a

total cost of 6.14 cents per yard, from cuts $\frac{1}{2}$ to 8 feet in depth. Most of this material was merely thrown to one side, while about 20 per cent of it was handled twice. The items of cost per cubic yard were made up as follows: Repairs while at work, 0.005 cent; annual repairs, 2.78 cents; fuel, 0.64 cent; wages and board, 1.85 cents; towing fuel, etc., 0.51 cent; superintendence, 0.3 cent; sundries, 0.03 cent. Total, 6.14 cents. The average amount handled was 96 cubic yards per hour, and about 10 per cent of the time was lost for repairs, moving, etc. During twelve months of 1909–1910 an elevator dredge on this river, using a belt conveyor for disposing of the material on the adjoining banks, removed 225,000 cubic yards of sand and clay at a total cost of $2\frac{8}{10}$ cents per yard.

In 1899 a contract was made for dredging a channel 1200 feet wide and 40 feet deep at mean low tide on the east side of New York Harbor, involving the removal of 22 million cubic yards of sand and mud, for 10 cents per yard (scow measurement). Two dipper and two suction dredges were used.

The contract price for removing 250,000 cubic yards of material from Raritan Bay, New Jersey, in 1902, from a maximum depth of 21 feet, was 16 cents per yard, scow measurement. The material was taken out to sea and dumped.

Other contracts made about the same time for removing sand, mud, and clay, varied from 35 cents per yard, scow measurement, for amounts of 5000 yards, to 30 and 20 cents for amounts of 30,000 to 72,000 yards, respectively. The material in most cases had to be towed to sea and dumped.

The gross cost per cubic yard of dipper and clam-shell dredging (in material other than rock) by the United States Government during the year 1910 varied from 3 cents to 22 cents, with an average price of about 10 cents.*

**Rock Dredging.**—Figures as to the cost of removing rock under water are indefinite, owing to the varying circumstances of each case. From a comparison we find that the minimum appears to be about $1.50, and the maximum about $17 per cubic yard, with occasional prices exceeding the last figure.

## SNAGGING.

In order to make navigation safe it is frequently necessary to remove obstructions such as snags, rocks, wrecks, etc., and to cut such overhanging trees as may interfere with craft. The term "snag" is used to describe stumps, logs, or drift caught or embedded below the water surface. This work is known as "snagging." Nearly all navigable rivers of importance in the United States have steam snag-boats, provided with suitable grappling and lifting appliances, explosives, tools, diving apparatus, etc., and these boats go over the rivers at the times most favorable for removing obstructions. The snags are sometimes cut or sawed into short lengths and placed upon the banks where

* For detailed costs see Part 3, Annual Report, Chief of Engineers, U. S. Army, 1910.

they will dry out and float off when a rise appears; sometimes they are boated to deep pools and dropped, when they sink to the bottom; and sometimes they are split up by dynamite, dried out, and burned.

The most rapid and economical method of disposing of dangers from snags consists in merely cutting them off below the depth required for navigation, the pieces removed being sawed into lengths of a few feet, so that they will not obstruct the river-bed elsewhere. This method is best adapted to small streams of stable bed, as those portions of the snags which are too deep or embedded too firmly to be removed can safely be allowed to remain without danger of shifting their positions. With sandy beds, however, such remaining portions are liable to be washed out and be moved away by the current, forming obstructions in new localities. In such cases the only safe method is to remove them entirely. If placed against the banks they should always be put upon the side that is building up (the convex bank), where they will often become in time permanently buried. If, however, the season is dry, they are liable to become light enough to float off when a flood comes, and becoming watersoaked, to sink further down the river, when they have to be rehandled. The surest method, though more expensive, is to haul all pieces (cutting up the largest ones where necessary) to the top of the bank by means of a block and fall placed high on a tree, which with heavy snags may have to be temporarily guyed back to a second tree. The height of the block should be such as to enable the pieces to be stood on end, and if a fire is lighted at the top of the pile after two to four months of dry weather, the pieces will usually burn until destroyed. If the fire is lighted at the bottom it will soon be put out by the drip from the logs. Where steam power is not available, recourse can be had to horses, mules, or oxen, the last-named being especially useful in spongy ground, because of their cloven hoofs.

On rivers of small depth where steam snag-boats could not move about during the low-water season, the work is done with tools and explosives carried on push-boats or bateaux. These boats are usually 10 to 15 feet in width, and 50 to 100 feet in length, and draw but a few inches of water. They are propelled by the crew by the use of poles. As they can move on a small depth of water, the low-water season is selected for the work, and it can then be done very effectively and economically, the snags all being in sight.

For rivers large enough to make a self-propelling boat advisable, a very convenient type is a stern-wheel steamboat, from 100 to 125 feet long in the hull, 24 to 28 feet wide (without any guards or side-overhang, as these are liable to be torn up in working along a bank) and $4\frac{1}{2}$ to 5 feet deep in the hull. The draft, when all is completed, will be from $2\frac{1}{2}$ to 3 feet. The boat should have at least two longitudinal bulkheads, trussed ones being preferable because of lightness, and be cross-braced also, and have a square or "scow" bow. It should be rigged with a pair of shears or preferably with a boom about 40 feet long, and an A-frame about 30 feet high, no mast being used. This length of boom is about the maximum with which large snags can be handled safely. The hoisting

engine should be of the 3-drum type, of 40 to 50 horse-power, two drums being used for the derrick falls, and the third one, which should be placed to one side, being used for pulling snags up the bank, for which purpose a $\frac{3}{4}$-inch steel wire line is employed. The propelling engines may be from 10 × 40-inch to 12 × 48-inch, according to the power desired. Sometimes a centrifugal pump, driven by the hoisting engine, is placed in the hull, and is used for dredging sand-bars, and an orange-peel bucket may also be added to the outfit. Timber and snag hooks, sling chains, etc., are provided as required, and a diving outfit is frequently very useful. Such a boat can usually be built and fitted out for a cost of $25,000 to $35,000.

A boat of this type (Fig. 59), as has been proved by an experience with them extending over many years, can be designed to handle any ordinary size of snag, and is suitable

FIG. 59.—Scow-bowed Steam Snagboat, Derrick-rigged.

to the majority of rivers. A larger boat is more expensive to build and operate, and will do little if any more work, and being usually of greater draft, cannot run in low stages when the work can be done most effectively. The boom derrick is preferable in most cases to the simple shears or A-frame without any boom (Fig. 60), as it can be made equally as powerful, and will dispose of the smaller snags more conveniently and quickly, and is available also for miscellaneous hoisting.

Figs. 61 and 62 show views of a snag-boat with a double bow, used on the lower Mississippi, where the depth of water is ample. The hull is of steel, 175 feet long by 62 feet wide by 8 feet depth of hold, with a displacement of 1100 tons. A crew of 40 officers and men is employed, and during 30 years' service the boat has given excellent satisfaction. The main hoisting chain is $2\frac{1}{2}$-inch, but snags are occasionally met with heavy enough to break it. A boat of similar type has also been used on the Ohio, but as the

depth of water is much less than on the Mississippi, its operations were frequently interfered with by its too great draft.

For rivers where physical or financial conditions preclude the use of a steamboat, but which require more thorough improvement than would be possible with the use of the push-boats before alluded to, an intermediate type of boat can be employed, an example of which is shown in Fig. 62*a*. The hull may be from 20 to 30 feet wide, 40 to 60 feet long, and $4\frac{1}{2}$ to 5 feet deep, the larger dimensions being preferable, as the boat can then do more effective work, besides affording more room for quarters, etc. A pair of shear-legs may be used for hoisting, as shown in the cut, or better still a boom as just described for the steamboat. The engine should be of the 3-drum type and not less than 30 horse-power. Quarters should be provided for the crew by an upper story,

FIG. 60.—Scow-bowed Steam Snagboat, Rigged with Shears.

as it is very desirable to have the boat self-contained. In some cases small removable paddle-wheels have been attached to the sides and have proved very useful for moving up or down the river; they are driven by a belt or by gearing from the engine, and arranged on projecting shafts which can be lifted out when snagging, so as to leave the hull free to get close to the bank. Steering can be done with a rudder or with a rafting oar, which consists of a plank fastened to a sapling and worked over the stern like a sweep.

There is one class of obstruction met with occasionally on streams flowing through a timbered country which is harder to deal with than any other. It is usually known in America as a "raft," and consists of a mass of drift and débris practically blocking the river and forming to all intents and purposes a fixed dam. The cause is the stranding of a floating tree or of pieces of drift on some obstruction, such as a snag, and as other trees and drift come down they are stopped in their turn, and being carried under by the current a blockade is gradually formed which may extend to the bottom

FIG. 61.—Double-bowed Snagboat "John N. Macomb." Employed on the Lower Mississippi River.

of the river. An example, partly removed, is shown by Fig. 62*b*. If the water continues to rise the mass may eventually be loosened and float off, but if the flood is receding a mat of trees and brush will be left which the river may be powerless to move. In such cases the mass will gradually fill with sediment; willows and trees will spring up upon it and tie it together still more firmly, and the water has to force its way through it and beneath it as best it may. In times of flood the river may spread over the adjoining lands, or may cut out a new channel at one side, and if much sediment is brought down a partial silting-up of the bed above is liable to result from the blockade. Such rafts have been met with where their surfaces had become level with the tops of the banks, and the mass extended 20 feet below to the bed of the stream. Examples existed on some of the rivers in Texas in 1904 with lengths of more than 20 miles, the upstream

FIG. 62.—View of Bow of the "John N. Macomb" (see Fig. 61) Showing a Snag Hoisted and Being Sawed Up.

ends growing with each flood, and on the Red River, a large tributary of the Mississippi, an immense raft was cut out between 1870 and 1880 which had stopped all navigation and was causing serious flooding of property. If the raft is of recent formation, a loosening of the lower end during high water may release the rest, but if it is of many years' formation the only sure way of removing it appears to be by using axe, saw, and cant-hook, cutting loose the pieces and burning them when dry, or pulling them on the bank. Dynamite and hoisting tackle will be of some assistance, but usually the pieces are so interlocked that they have to be cut up before they can be removed.

On some rivers, as the Ohio, wrecks of barges are a frequent cause of obstruction. The large tows occasionally become unmanageable and strike bridge piers or other obstacles, and some of the coal boats and barges are sunk. Their speedy removal is frequently necessary, particularly in the upper part of the river, where the coal rises

FIG. 62*a*.—Snagboat of Barge Type, with Shears.

FIG. 62*b*.—Driftwood or "Raft" on the Upper Trinity River, Texas, in Process of Removal.

pass off quickly, and the channel must be opened before the navigable depth has passed. Dynamite is used for this purpose as far as practicable, and sometimes dredging is resorted to. Coal boats have thus been removed within a few hours after sinking, so that tows could pass without delay or danger.

**Cost of Snagging.**—The cost of snagging a river varies within wide limits, being usually the highest on small streams. Banks on which many trees have to be felled and cut up can usually be cleared for an expense ranging from $100 to $200 per mile for timber cutting only. Where the snags are cut off to low water and left in place, the cost of such work may vary from $30 or $40 to $200 per mile, and where they have to be pulled out and cut up the cost may run as high as $500 or more per mile. If, in addition, there are occasional boulders or small reefs to be blasted out, the cost may reach $800 or more per mile. The above figures do not include the cost of plant, and refer only to the original work. After obstructions have once been removed, the cost of maintenance is comparatively light, a crew of 10 to 20 men with suitable equipment being usually able to care for 200 or 300 miles of river, working three or four months a year. In wet seasons, however, the cost is liable to be considerably increased owing to keeping the force under pay until the subsidence of the freshet allows work to be begun again.

On the Mississippi River between Cairo and New Orleans, a distance of about a thousand miles, the cost of snagging from 1872 to 1910 was about $2,800,000. During that period nearly 100,000 snags and about 100 wrecks were removed, and about 450,000 trees were cut from the banks.* This cost in round numbers is equivalent to about $74,000 a year for the 38-year period, or to about $74 per mile.

* Annual Report, Chief of Engineers, U. S. A., 1910.

## DIMENSIONS OF TYPICAL DREDGES.*

### SEA-GOING SUCTION DREDGES.

| Name. | Where Employed. | When Built. | Material of Hull. | Length Over All. | Beam. | Depth. | Draft, Light. | Capacity of Hopper. | Displacement, Loaded. | Horse-power. Propelling Engines. | Horse-power. Pumping Engines. | Dredging Pumps, Number and Size. | Working Pressure. | Average Capacity per Hour. | Depth. Before Dredging. | Depth. After Dredging. | Total inclusive Cost of Dredging per cu. yd. | Cost of Dredge. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Feet. | Ft. Ins. | Ft. Ins. | Ft. Ins. | Cu.yds | Tons. | | | No. Ins. | Lbs. | Cu.yds | Feet. | Feet. | Cents. | Dollars. |
| Galveston..... | Galveston harbor......... | 1908 | steel | 304 | 51 0 | 28 0 | 13 6 | 2850 | 7290 | 1900 | 1000 | 2 20 | 145 | 701 | 31 | 33¾ | 6.07 | 357,500 |
| Clatsop....... | Portland, Ore............ | 1908 | " | 180 | 38 0 | 23 0 | 17 6 | 1061 | 3158 | 900 | 500 | 2 18 | 135 | 964 | 20 | 26 | 4.11 | 234,500 |
| Delaware...... | Delaware River.......... | 1905 | " | 315 | 52 0 | 22 6 | 12 6 | 3147 | 6440 | 1426 | 630 | 2 20 | 118 | 1127 | 25–30 | 30–35 | 2.48 | 358,400 |
| Sumpter....... | S.&S. W. Passes, Miss. River | 1904 | wood | 200 | 41 0 | 20 0 | 13 9 | 990 | 2664 | 645 | 600 | 2 18 | 115 | 447 | 21 | 30 | 18.35 | 191,000 |
| Winyah Bay... | Charleston harbor........ | 1898 | " | 141 | 31 6 | 13 5 | 9 8 | 300 | 833 | 355 | 208 | 1 15 | 110 | 288 | 19–21 | 23–26 | 15.45 | 73,800 |
| G. B. Crow.... | Mersey.................. | 1895 | steel | 320 | 46 10 | .... | loaded 16 6 | 2000 | ... | ... | 700 | 1 45 | ... | ... | .... | .... | ..... | 291,000 |
| **PIPE-LINE HYDRAULIC DREDGES** | | | | | | | | | | | | | | | | | | |
| Vesuvius...... | Galena River............ | 1909 | wood | 120 | 26 0 | 5 0 | 2 10 | ... | 209 | ... | ... | 1 15 | ... | 157 | 2½ | 8.0 | 2.27 | 30,000 |
| Pettus †....... | Alabama River........... | 1909 | " | 135 | 35 0 | 6 6 | 4 3 | ... | 504 | ... | ... | 1 15 | ... | 143 | 0½ | 6.0 | ..... | 62,400 |
| Pascagoula †... | Pascagoula River......... | 1909 | steel | 150 | 40 0 | 10 6 | 5 3 | ... | 794 | ... | ... | 1 20 | ... | 341 | 13.2 | 20.2 | 5.35 | 143,600 |
| Barnard....... | Gulfport harbor.......... | 1904 | " | 206 | 38 0 | 14 6 | 10 0 | ... | 754 | ... | ... | 1 36 | ... | 1319 | 17.2 | 22.0 | 3.67 | 230,000 |
| Beta.......... | Mississippi River......... | 1896 | " | 214 | 58 0 | 6 9 | 4 0 | ... | ... | ... | 1850 | 2 33 | 175 | 1600 | .... | 36.0 | ..... | 172,800 |
| B. M. Harrod.. | Mississippi River......... | 1907 | " | 210 | 44 0 | 8 6 | 6 6 | ... | ... | ... | ... | 2 24 | 138 | 1560 | 9.0 | 15.0 | 1.00 | 239,000 |
| J. Israel Tarte † | St. Lawrence River....... | 1902 | " | 160 | 42 0 | 12 6 | 6 0 | ... | ... | ... | 980 | 1 36 | 150 | 2600 | 35.0 | 50.0 | ..... | 163,800 |

* Chiefly compiled from the Annual Report, Chief of Engineers, U. S. Army, for 1910, Part 3.
† These dredges have cutters.

CLAMSHELL, DIPPER, AND ELEVATOR DREDGES.

| Name. | Where Employed. | Type. | When Built. | Material of Hull. | Length Over All. | Beam. | Depth. | | Draft, Light. | | Displacement. | Capacity of Bucket. | Length of Boom. | Working Pressure. | Average Capacity per Hour. | Average Cost of Dredging per cu. yd. | Cost of Dredge. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Feet. | Feet. | Ft. | Ins. | Ft. | Ins. | Tons. | Cu.yds. | Feet. | Lbs. | Cu.yds. | Cents. | Dollars. |
| No. 1........ | Galveston............ | Clamshell | 1908 | wood | 65 | 27 | 5 | 6 | 3 | 0 | 115 | 1½ | 48 | 100 | 100 | 15.00 | 8,800 |
| Hercules..... | Cape Fear River....... | " | 1907 | " | 100 | 38 | 11 | 4 | 7 | 0 | 670 | 7 | 55 | 90 | 195–290 | 4.20 | 64,500 |
| Guadalupe.... | Guadalupe River...... | " | 1908 | " | 118 | 29 | 5 | 9 | 4 | 0 | 303 | 1½ | 46 | 150 | 100 | 14.18 | 38,100 |
| Cheraw...... | Gt. Pedee River....... | Clamshell and dipper | 1906 | steel | 150 | 28 | 8 | 0 | 4 | 6 | ... | 3 cl. 1¾ dr. | 58 | 100 | 240 | 88.0 | 45,500 |
| Tennessee... | Tennessee River...... | " | 1910 | wood | 80 | 34 | 6 | 10 | 4 | 4 | 398 | 1½ cl. 2 dr. | 44 | 125 | 200 | 6.00 | 37,500 |
| Frankfort.... | Kentucky River....... | Dipper | 1908 | " | 72 | 19 | 5 | 5 | 3 | 6 | 128 | ¾ | 27 | 125 | 50 | 22.84 | 10,200 |
| Maumee..... | Lake Erie............ | " | 1909 | " | 100 | 36 | 11 | 0 | 7 | 6 | 533 | 3½ | 42 | 70 | 150 | 22.50 | 42,000 |
| Green River.. | Green and Barren Rivers | " | 1896 | " | 112 | 31 | 4 | 0 | 2 | 9 | 298 | 1¼ | 60 | 110 | 50 | 16.00 | |
| Malta....... | Muskingum River..... | Elevator | 1888 | " | 70 | 31 | 6 | 10 | 3 | 6 | 160 | 24 of 3 cu.ft. | ... | 100 | 120 | 9.85 | 17,000 |
| Alabama..... | Muscle Shoals......... | " | 1891 | " | 80 | 38 | 6 | 6 | 3 | 6 | 261 | 24 of 5 cu.ft. | ... | 96 | 100 | 3.60 | 22,600 |
| Volgskaga.... | Volga............... | " | 1906 | steel | 157 | 31 | 9 | 8 | 5 | 0 | 775 | .... | .... | 150 | 326 | .... | 23,100 |
| Baldwin...... | St. Lawrence River.... | " | 1902 | " | 165 | 34 | 14 | 0 | 8 | 0 | .... | .... | .... | .... | 300 | .... | ...... |
| Corozal...... | Panama............. | " | 1911 | " | 269 | 45 | 19 | 0 | 14 | 4 | ... | 34 & 54 cu.ft. | ... | 180 | 1200 | .... | 399,300 |

## CHAPTER IV.

### DIKES.*

**General.**—The structures used for guiding a stream by confining and directing the water at bars or shoals, and for closing secondary arms of a river, are all known under the general name of dikes. The earliest known example of their employment is found on the Nile, where a system of stone spur and longitudinal dikes was built by the Pharaohs for the purpose of reclaiming land. Many of these structures are still in existence.

In the succeeding chapter a description will be given of the types of dikes used to protect banks from erosion, and of the usual methods of employing them. The present chapter treats of dikes for improving channels for navigation.

In the chapter on Regulation were given the general principles, summarized in the rules of M. Fargue (p. 51), which observation has shown to govern the flow of rivers. The object of dikes, when used for the benefit of navigation, is to modify the low-water flow according to these laws as far as circumstances will allow, and to make it conform to a bed of greater regularity, so that the depth, width, and curvature may be utilized to the best advantage. Thus where the natural channel is too wide and therefore too shallow it has to be contracted and made to become deeper; where the curvature is too sharp it must be made less abrupt; and where sudden or irregular changes of depths are found they have to be modified to secure a better distribution of the water. The influencing of the high-water flow by means of dikes is an object rarely aimed at for the sake of navigation, since in times of rises the depths are usually more than sufficient, and it is only when the water falls that shoals or irregularities of channel begin to present obstructions.

The most common use of dikes is to contract the channels over shoals; their use as a means of correcting curvature or reducing excessive depths is less frequent. Briefly recapitulated, the effect of a flood (see pp. 7, 13, and elsewhere) is to deposit sediment on the shoals, which are gradually scoured down to their normal low-water height as the flood recedes. This process of scouring, however, is gradual, and navigation may be delayed by insufficient depths while it is in progress. On many rivers also the natural depths are not enough even when the river has removed the excess material, and in such cases dikes are often employed to contract the flow at this period, and thus hasten the

* Some of the plates accompanying this and the next chapter are republished by permission from "The United States' Public Works' Guide and Register," by Captain W. M. Black, Corps of Engineers, U. S. A., and from the papers by H. St. L. Coppée on "Bank Revetment," published in Transactions American Society of Civil Engineers, January, 1896.

scour, while providing at the same time greater depths from narrowing the space through which the water has to pass. In many rivers dredging is employed in lieu of or in connection with dikes, as described in the last chapter. In Germany, on the upper Mississippi, the Rhone, and elsewhere, considerable lengths of a wide, shallow river-bed have been reduced by dikes on both sides so as to concentrate the flow and provide navigable low-water depths.

It is frequently very desirable during the construction to employ dredging to assist the scour, not only because more rapid results are obtained—the water tending to follow and scour in the trench dug—but also because the surface material of a shoal, especially at its head, is often heavier than that underneath, and once the covering is removed the scour proceeds more rapidly. The material removed by the dredging is usually placed behind the dikes so that as little of it as possible may be washed back into the channel. On some shoals, moreover, there occur hard places, such as spurs of clay, deposits of gravel, etc., which produce irregularities of channel which the increased velocity of the water will be unable to correct. As long as these remain the channel will be unsatisfactory, and dredging should be resorted to in order to assist the action of the current. Lastly, the contraction of the channel forces upon the river the burden of disposing of the scoured material, and if this is large in quantity there will result more or less disorganization of the bed below, with perhaps a tendency to considerable changes in the channels and the banks. Where dredging can be employed to remove the material, thus minimizing the work entailed upon the water, the risk of troubles below will be largely avoided. On shallow rivers this dredging can often be done by a grapple or clamshell bucket attached to a derrick boat, the material being dumped behind the dike.

After the completion of the work it will be found that the new material brought upon the shoal by floods is usually lighter than the old, and thus the scouring will take place more easily. The general deposits will occur as before, since they are due to the slackening of the flow during floods by reason of the wider cross-section of the bed, whose total area is but slightly affected by the contraction of the dike. However, as the flood recedes the effect of the latter commences, and its full influence is in play when the water has reached the level of its top. The newly deposited sediment is thus scoured away more quickly than would have been the case before the improvement, and the delay to navigation is greatly lessened and often entirely removed.

**Amount of Contraction.**—The general relations between the supply of water, the dimensions of the channel and the material of the bed, have been described in Chapter II (p. 64 and after). The success of the improvement will depend chiefly upon the securing of a suitable amount of contraction and a proper location for the dikes, both of which questions are difficult to decide.

It has been explained (p. 68) that when a channel is contracted at a shoal, the increased velocity will tend to scour away the material and deposit it below until a slope has

been obtained which will reduce the speed and permit the material to again rest undisturbed. This action is illustrated by Fig. 63. When a receding flood falls to the level of the top of the dike or dikes, the width of flow becomes suddenly reduced, and the discharge, while still considerable, is contracted into a comparatively narrow channel. At this time, owing to this contraction, the scouring away of the material which the flood has deposited on the shoal is proceeding rapidly, but by the time the ordinary low-water discharge has been reached the scouring should have ceased, after having produced a channel deep enough for boats and without having lowered the water surface of the pool above. It is evident, therefore, that whatever low-water depth may be desired for the new channel, the amount of contraction will have to be adjusted to suit the material unless the bed is held by groundsills. If it is made too great, the water will scour too much and there will result a lowering of the water in the pool above, accompanied by a steepening of the slope at the shoal upstream and a consequent scour upon it; if the contraction is too small, the velocity may be insufficient to complete the scour, when the depth will again

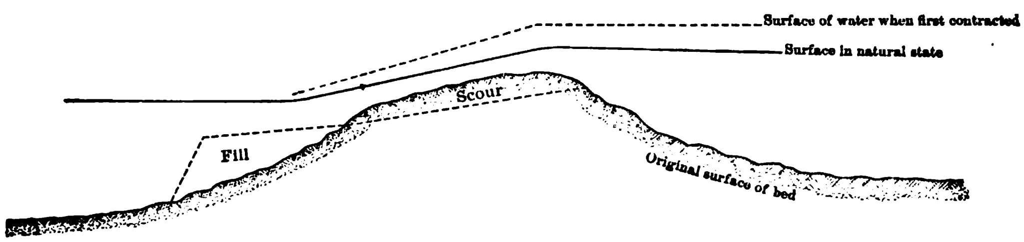



FIG. 63.

be unsatisfactory. The outline of the new channel (expressed by its hydraulic radius), being more favorable to ease of flow under a given slope than was the former outline, will tend towards an increase of velocity, and will thus cause scouring under conditions of slope which before gave equilibrium. A balance will again be obtained only when the effect of the new cross-section has been offset by a proportionate flattening of the slope.

To apply the formulas given on p. 71, the velocity and the slope at the unimproved shoal are first determined for ordinary low water (when scouring has usually ceased), as these factors will give some idea of the limits at which the material ceases to be moved. Having next determined the best location for the dike and the probable elevation of its top above low water (see "Height of Dikes," p. 149) the cross-section of the new channel is approximated for various points, and the formulas are then applied so as to trace as far as possible the theoretical effects of the improvements. The areas of the new cross-sections will depend principally upon the discharge, the slope, and the hydraulic radius, as combined by formula (1), etc. The discharge is known, being that of ordinary low water, but

the slope cannot be predicted with exactness. It will in all probability be somewhat less than the original, for reasons given above; how much less can only be approximated. The finding of the new hydraulic radius is also a matter of approximation, and the general outline of the new channel will depend largely on the relative positions of the dike and the axis of flow. If the latter is close to the dike, as in a bend, the channel will tend to assume an outline as in Sections *AA* or *BB* of Fig. 2, p. 14; if in a crossing, the outline will approximate that of Section *BB*, Fig. 65, p. 141.

The amount of contraction is thus always difficult to decide, and may vary somewhat from shoal to shoal, because of changes in the discharge and slope, the nature of the material, etc. Calculations will afford some guide as to the dimensions of the channel, but the precise effect of the water upon the bed cannot be foretold, and the construction should proceed in a manner that will admit of modification if found necessary to suit the changing conditions.

It must be borne in mind that unless sufficiently deep pools exist below the shoals in which the eroded matter can be deposited and thus be out of the way of navigation the scouring action will be greatly retarded and perhaps produce no useful effects. (See p. 71.) If such pools are wanting the shoals will merely increase slowly in length and there will be a tendency to obliterate the few deep spots which may be in existence. In such cases dredging may be of some use, but its effects will probably be only temporary and will have to be reproduced after each flood. On rivers of this regimen dikes may prove of little use as a means of improvement, unless a considerable and permanent contraction can be made and the bed held by groundsills, the depth of water being thus increased while the bottom is kept at the original level. Where these deep pools exist, they act as receptacles for the scoured matter, and hold it until the next flood, when the water, in accordance with the principles explained in Chapter I, takes up as much of it as it can and bears it down the river, perhaps only as far as the next shoal. Here it may cause other complications, but eventually it will be carried out of the mouth of the river.

Some idea as to the limit of the velocity permissible at a particular locality can be obtained by taking a specimen of the material and subjecting it to experiments, in order to find out at what speeds scour will begin. The indications, however, are only approximate, since there may exist considerable variations in the composition of the bed itself, and still other causes may modify actual results. Thus at one locality on the Brazos River, Texas, it was found that the shoal under improvement, which apparently consisted entirely of sand, scoured only to a certain point. On examination it was found that occasional pieces of small gravel had been scattered through the bed, and that these had gradually collected upon the surface as the sand was scoured away until they formed a covering sufficient to stop further erosion.

A more reliable guide as to the amount of contraction, or the size of the channel,

can often be found by an examination of the river in the neighborhood, where in the straight reaches just above or below the shoals conditions are often met with which display approximately the dimensions wished for. Thus suppose Fig. 64 to represent a shoal and that the channel is to be 4 feet deep. Suppose the normal low-water cross-section of the shoal, as at *CC*, has a total of 450 square feet with an average depth of 3 feet, and that at *AA*, approaching the shoal, the total is 600 square feet with an average depth of 6 feet. At some point between *AA* and *CC*, as at *BB*, there will be found a cross-section approximating the 4 feet depth of the channel wished for, and indicating at the same time to what width the river should be contracted in order to produce about that depth. Similar cross-sections usually exist at the lower end of a shoal. If they are not found either above or below, owing to the local peculiarities, the indications can generally be obtained from shoals in the neighborhood. Taking this as a basis, the new area may be made 10 or 15 per cent less than that at *BB*, if the new conditions are to be produced by scour alone, and without the help of dredging. If a dike be constructed as at *DD*, gradually reducing the width until the crest or shallowest portion is reached at *CC*, the conditions at *BB* will be approximately reproduced down the shoal. The velocity and slope at *BB* in the natural state are less than at *CC*, and when the dike is built the relative velocities will be still more unequal. However, the greater speed is needed at *CC* in order to produce the scour, and as this progresses the slope becomes flatter and the conditions approach more nearly those at *BB*. It is wise where practicable not to build such a dike of full length, nor to make it impermeable or continuous until several floods have occurred to give an indication of its probable effect. By this means the final construction can be made suitable to the evolving conditions, which should be carefully examined from time to time.

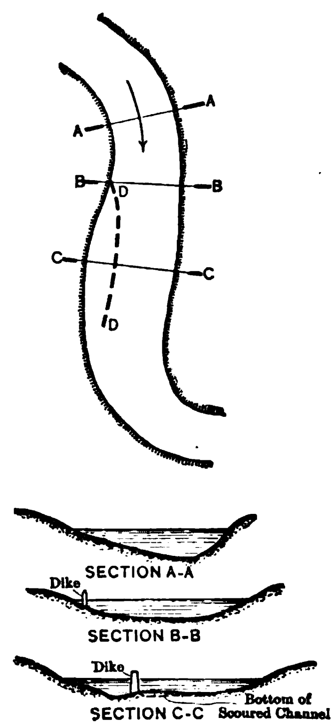



FIG. 64.

Experience appears to have shown that where an improvement of shoals subject to erosion has changed the natural conditions so as to increase the velocity even in a moderate degree, without groundsills being used to limit the scour, the undesirable results before described have invariably followed sooner or later. At first the channel becomes deeper and navigation is improved, and this benefit may extend over two or three seasons or longer. Gradually, however, as was instanced by the "Canal de Miribel" (p. 69), the low-water surface begins to fall, new obstructions are uncovered and the shoals upstream begin to scour, new bars are formed downstream by the scoured material, and the last condition becomes worse than the

first.* In many cases groundsills have had to be constructed later in order to stop the derangement. (See "Submerged Spurs or Groundsills," p. 155). Where these are used in the original construction the scour can be controlled within definite limits, and the amount of contraction desirable can be closely approximated.

If dredging alone is used to obtain more depth through a shoal, there still follows the tendency towards an increase of velocity and an ultimate change in conditions as described above. This ceases, however, when the next flood arrives, as the dredged channel becomes filled with deposit and the natural conditions are thereby restored. The tendency, in other words, is intermittent, and does not continue long enough to produce any permanent change.

**Location of Dike.**—Having ascertained the cross-section or size to be used for the contracted channel it remains to determine the proper location for the dike. In all rivers except those of very unstable bed it will be found that the water approaches and passes over the shoal in a direction which is usually more or less constant and which the passage of floods changes but slightly if at all. This direction is chiefly dependent on the contour of the banks just above, since these in high water modify the discharge upon the shoal and reproduce at every flood with an approximate sameness the eddies and slackwater, and therefore the deposits. These tendencies remain constant as long as the banks are unchanged, so that after the dikes are built the high-water deposits will form at about the same places as before, although they will be removed more quickly. The improvement should thus be based as far as practicable on an evolution of the existing channel rather than on the creation of a new one; the natural tendencies towards easy flow should be assisted as far as practicable, rather than made to change unduly their direction.

Shoals occur either in those portions of a river where the axis of flow crosses from one bank to the other or in long reaches where the current seems to have lost directness. In either case the shoals are due, in the great majority of instances, to a superfluous width of bed, which in addition permits the flow in low water to spread out and become shallow. Occasional examples are met with on every river, however, where these conditions are not found, and where the width at the crossing does not become excessive and a good channel therefore exists at all seasons of the year. (See p. 15). These examples confirm the observations and rules set forth by M. Fargue (p. 51), that in order to obtain a greater uniformity of channel the artificial bank lines should be laid out so as to contract the flow gradually as the crest of the shoal is approached, and should direct the current with easy alignment so that it will cross from deep water above to deep water below. These dispositions would reproduce the conditions usually existing with naturally good channels, and an example of their application on a large scale will be found

* See reports of MM. Jacquet and Pasqueau on the improvements of the Rhone and the Saône, published in "The Improvement of Non-Tidal Rivers," by Col. Wm. E. Merrill, U. S. A.; also "Rivières à Courant Libre," by F. B. DeMas, and elsewhere.

in the description of the regulation of the Rhone (p. 72 and after). Fig. 65 indicates the general theoretical outlines of the method and further illustrations and some applications are given in Figs. 11 to 25, p. 55 and after. It is rarely necessary, however, to construct a continuous artificial bank for each side of the proposed channel.

Experience has shown that two important points should be observed in the location of the dikes: the one, that the natural tendency of flow—that is, the direction the water would take if unobstructed—should be preserved as far as practicable, and the other that it should impinge on the dike line at a very flat angle and be carried along it with an easy alignment. If the dike crosses the flow entirely it will, in all probability, cause the production of a scour hole and an irregular channel, and if it impinges on the flow too suddenly similar effects may result. Thus suppose Figs. 66 and 67 to represent shoals, one with a " middle ground " or center shoal and the other without, the axes of the low-water flow being as shown by the arrows. If a dike is projected far into the river bed as at *AA* so that it breaks into the line of flow, scour holes will almost invariably appear near the places shown, accompanied by a local narrowing of the channel and perhaps a danger of undermining that part of the structure. In the case of Fig. 68, if the dike be composed of straight lines and either of the points *C* or *D* encroach too much upon the flow, scour holes will again appear of a size proportionate to the amount of interference. A sharp angle as at *C* is in any case not desirable, as the shore side of the current will be deflected too suddenly and produce an eddy at the angle. The line should approach gradually as shown in Figs. 66 and 67. The scouring action is similar to that produced by a jet of water played upon a bank; the greater and the more sudden the impingement the greater will be the erosion. Pls. 10 and 11, which show dike improvements on a river in the Mississippi basin, afford an illustration of certain of these principles.

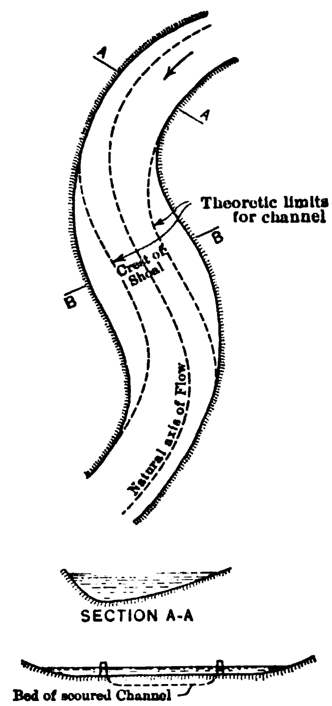


FIG. 65.

The controlling dike line, therefore, according to what has preceded, should be located so as to assist the water in finding an approximately direct path from deep water above the shoal to deep water below (see also p. 18); it should approach and meet the current on an easy curve, and so that the maximum contraction will occur at the crest of the shoal; and it should retain the water in the channel until the flow has passed below the crest to a distance which will ensure at all times a sufficient depth. The actual location of the line, however, is a matter largely dependent on circumstances, and the general principles laid down have usually to be modified to suit local conditions

as well as the funds available. Thus a thorough improvement of the shoal in Fig. 66 might require the new channel to follow a line of current as shown by the arrows in Fig. 67, but a more economical improvement and one possibly as satisfactory might be secured by modifying one of the two existing channels. (See also p. 53 and after.) The amount which the dike line may encroach upon the channel must also be approximated from a study of the local currents and of the bed, indications being usually obtainable from the cross-section of the channels near by, as previously described. Lastly, the length of dike line should be determined as the effects of the work develop, and it should be made too short at first if the proper length is a matter of doubt, since this can be increased later if necessary.

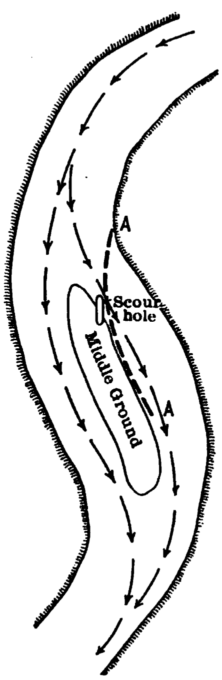


FIG. 66.

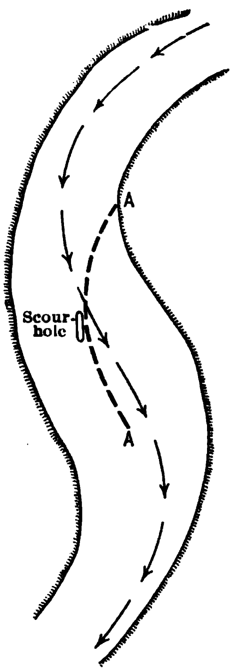


FIG. 67.

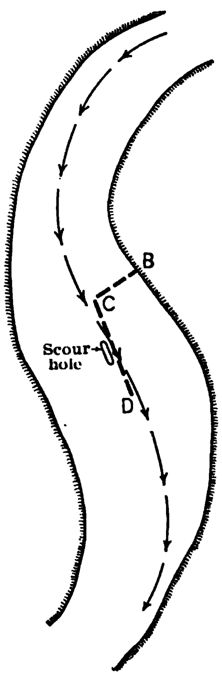
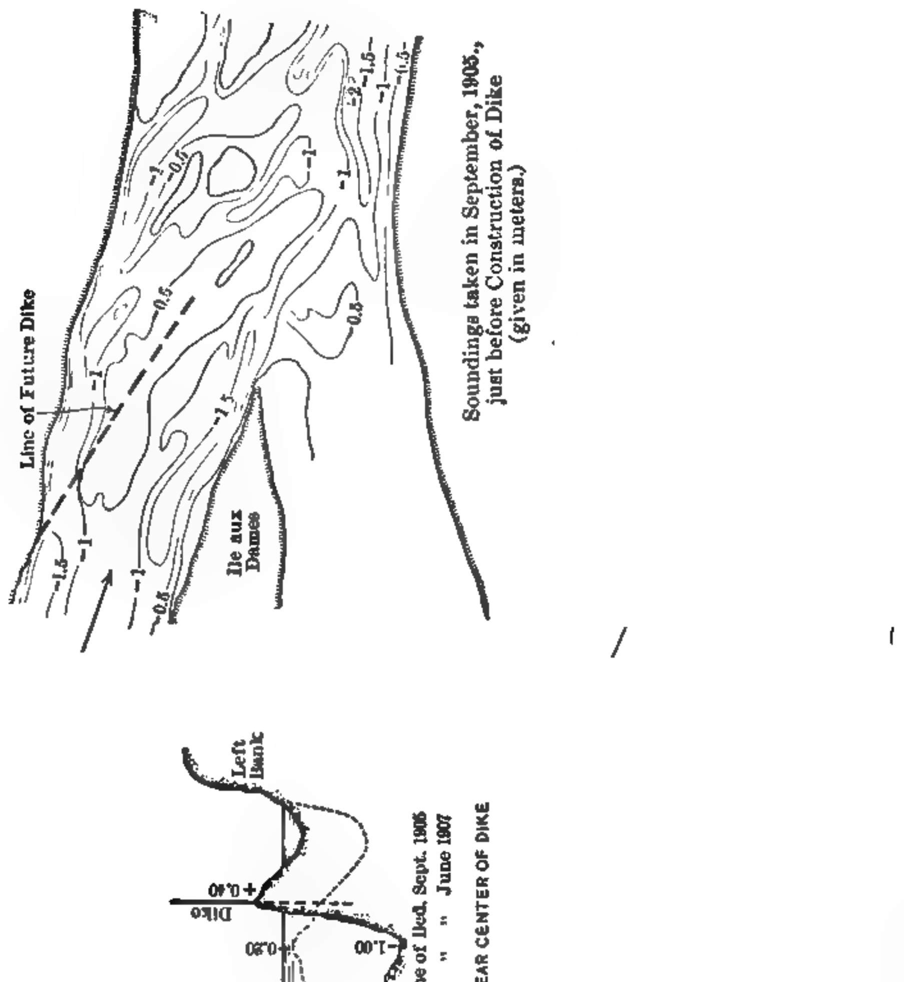


FIG. 68.

In regard to the side of the channel on which the dikes should be placed, the convex or "making side" is often the better, since the work is better protected from drift and less subject to undermining. In many cases, however, the other or concave side has to be chosen and the dike made to serve at the same time as a protection for the bank. This function is not required when the work is placed on the convex side, since this builds out naturally in proportion as the concave bank erodes, but if the dike projects too far into the river it will increase the erosion on the concave bank and as the bed widens it will begin to regain its former shallowness.

**Choice of Type.**—The general types of dikes in use are the longitudinal dike (frequently called the training wall); the spur-dike; the submerged spur or sill (also called the groundsill); and the closing dike or dam. The first two are the only ones used

directly as training works, the submerged spur and the closing dike being employed chiefly to modify and control the depth of flow.




Soundings taken in June, 1907.

IMPROVEMENT OF THE LOIRE AT PORT THIBAUT BY A LONGITUDINAL DIKE.
(FITTED WITH SLIDING GATES TO MAKE IT PARTLY PERMEABLE)

FIG. 68a.

Broadly stated, and neglecting the question of comparative cost, the longitudinal dike is preferable where the structure is on the concave side or where the flow is small, since this type affords a smoother passage for the water and does not create the eddies or irregularities of depth usually produced by spurs. (See Fig. 68a and Pl. 12.) On

the other hand it is more difficult of modification than the latter; if it is found to be placed too far out upon the shoal, or not far enough, any change will be costly to make, whereas a spur-dike can usually be lengthened or shortened without great expense. An objection sometimes urged against longitudinal dikes is that at certain stages the surface currents pass obliquely over them and tend to draw boats against the crests. The same holds good, however, against spur-dikes in similar positions, and on the Rhine, before the spaces between the spurs had become filled, much caution had to be used by boats in passing certain localities in order to avoid being drawn in upon the structures. The longitudinal dike is more exposed to undermining from currents along the river face, and in exposed locations the water pouring over the crest at the upper end will tend to undermine the inner side also.

The spur-dike appears to produce the best results when used on the convex bank, or where the currents are moderate, as on a river of slight fall or in tideways.* Under such conditions there is very little tendency for the movement of the water to cause erosion in the pockets between the dikes, and these in consequence provide valuable spaces for the deposit of silt by natural means. There are two general types, the solid spur and the permeable spur, the latter being of a construction which permits a portion of the flow to pass through it, thus slackening the velocity and causing much of the sediment to be deposited. The permeable spur is usually composed of less permanent materials than can be used for solid spurs, but it causes practically no eddies and can thus often be used on the concave side; it is less exposed to undermining, and the filling out of the bank line is usually rapid and permanent. The eddies or scour caused by solid spurs when built in strong currents or on the concave banks can be traced in many of the Rhine dikes (Pls. 3, 4, and 5), on those of the Rhone (Pls. 1 and 2), and on Pl. 12, which shows the effects of a series of spurs on a shoal in a river in North Carolina. The tendency of this type when thus used in rivers of small discharge is to produce an irregular channel, since they do not possess sufficient continuity to make the flow keep a straight course, and the water in trying to get back to its natural low-water channel will pass in and out as shown in Fig. 69. In high water the obstructions act like submerged dams, and usually cause horizontal eddies until the water reaches the tops and vertical eddies at the higher stages, producing erosion, as shown on Pl. 12. The spur-dike, if on the concave bank, is also more liable to injury by drift and ice than the longitudinal dike, as the latter presents a more even surface to passing débris.

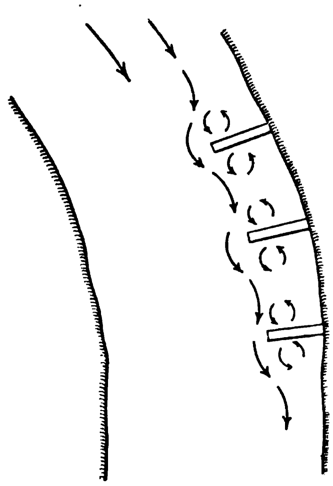


FIG. 69.

* See also "Spur-dikes for Bank Protection" in the next chapter.

The method of spur-dikes has had an extended application in Germany on the Rhine, Elbe, Vistula, Oder, and other navigable streams, and has been employed in several other countries, including the United States. In this method the dikes are placed at intervals along the shores, and project more or less into the stream, either normal to the channel or slightly inclined to it. While their chief object is the improvement of navigation, the deposit of alluvium between the various spurs may become of considerable importance, and under favorable conditions these deposits may reach the level of the tops of the dikes and a new and continuous bank will thus be formed. It is stated that 8400 acres of river-bed on the Prussian part of the Rhine was thus transformed into alluvial soil.

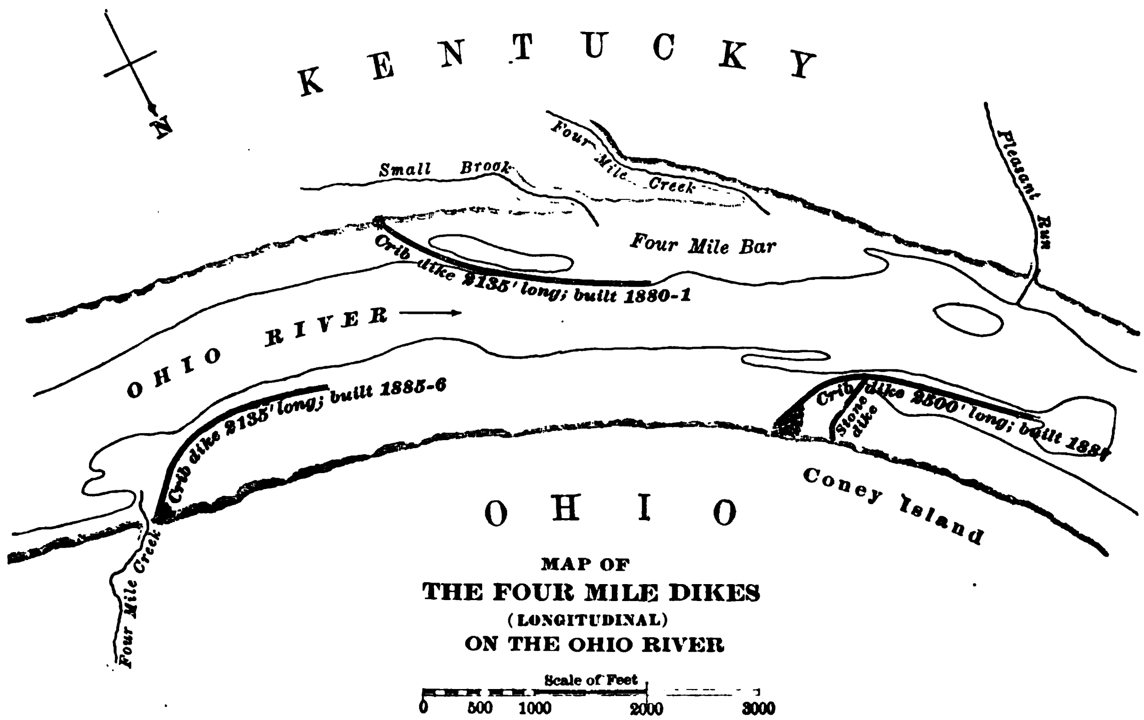



FIG. 70.

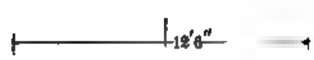
A description of the spur-dikes used in America for protecting banks will be given in Chapter V, and a more detailed account of those on the Elbe will be found in paragraphs further on.

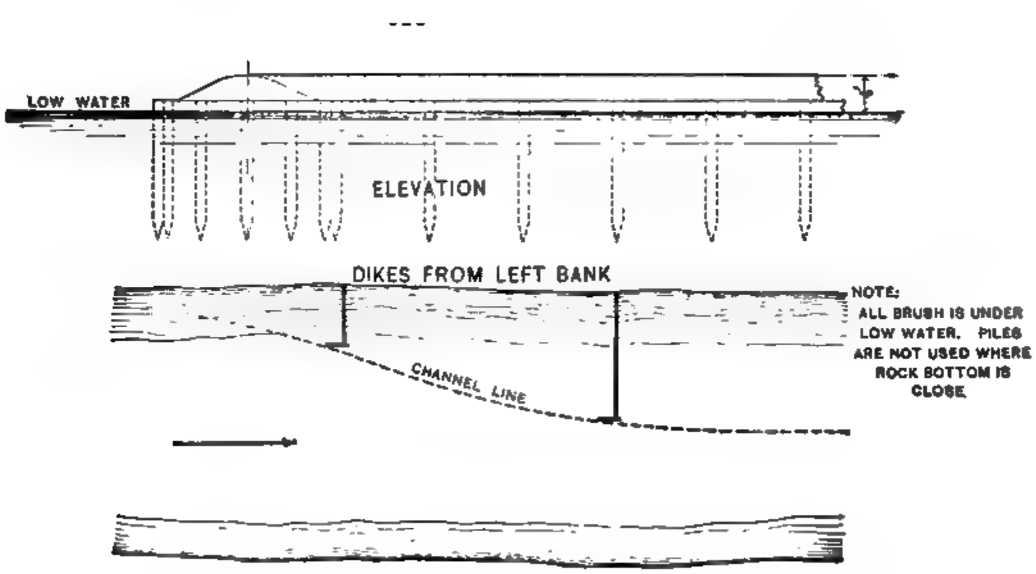
Longitudinal dikes, which are built about parallel to the bank and to the direction of the current, have also been extensively used in Europe, and have met with considerable favor in this country. Those used upon the Ohio River (see accompanying cuts and Fig. 70) are typical examples. Starting from the bank they describe a continuous curve downstream until their direction becomes parallel to that of the proposed channel, when they follow the line of the latter. At ordinary flood stages their crests are submerged so that boats may pass over them in safety. The spaces behind some of them

gradually fill with sediment and even grow up with willows until a new bank is formed, while behind others a good deal of scour takes place, due to the water overflowing the dike and tending to undermine it on the inner side.

The German engineers appear to give preference to the spur type, while in France the longitudinal dike has been the more favored. It is to be noted, however, that the improved rivers of Europe are generally of a much milder regimen than those in the United States, and are not subject to such high and rapid floods, nor do they carry a similar proportion of sediment or drift. Under such conditions the relative effects of the two types of dikes would be less pronounced. The spur-dike has been used with much success in America on wide rivers, such as the upper Mississippi, and on the Missouri the permeable spur has proved both economical and effective. An engineer of wide experience with dikes on smaller streams has informed us that in his opinion, and neglecting the questions of comparative expense and special local conditions, longitudinal jetties were preferable for rivers with high banks, short low-water seasons, and high floods, as they appeared to cause an earlier and more rapid scour by their uniform contraction of the flow. The scour produced by spur dikes did not seem to commence until the water had about fallen to the level of their tops and at a later period than with the other type, while in high water, if the solid spur was used, objectionable erosion took place between the structures. The resulting low-water channel was thus liable to be irregular and as shown in Fig. 69 and Pl. 12. For rivers with long seasons of low-water, however, spurs frequently proved effective, and especially so in tidal reaches, where the currents were moderate and the spaces between the dikes would in consequence gradually silt up and create a new bank line. The distance between the spurs on small rivers should not, in his opinion, exceed 200 feet.

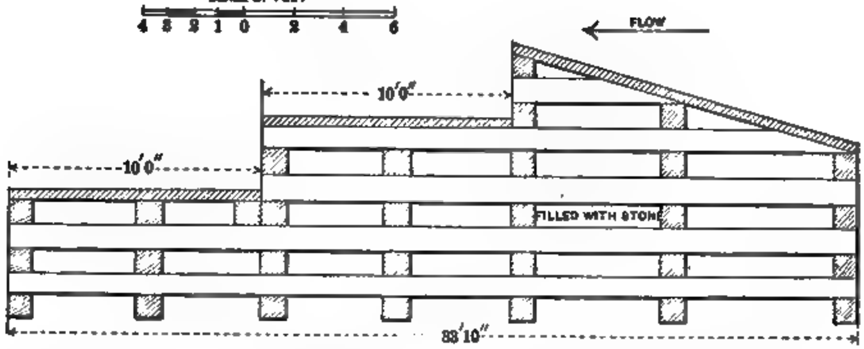
**Spacing and Alignment of Spur-dikes** (See also "Spur-dikes for Bank Protection" in the next chapter).—The spacing or distance between spur-dikes varies with the width of the channel, and is dependent also to some extent on whether they are placed on the convex or concave bank. If on the latter they should have a closer spacing than on the former, so as to reduce the tendency of the current to wind in and out between them. No fixed rules appear to be available for determining the distance, as too many considerations, natural and economical, enter into the problem. The closer the dikes are placed, however, the more regular will be the channel. It is stated that on the Elbe the spacing on concave banks was made one-half, and on straight reaches about nine-tenths, of the channel width. On the Memel the spacing on straight reaches was five-sevenths of the normal channel width, while on concave banks it was made one-half, and on convex banks equal to the channel width. In the latter locations the distance varied somewhat with the length of the dikes, and where they were short it was made about equal to their length. On the lower Rhine the distance between spurs varied from 325 to 500 feet, and on the Waal, one of the outlets of the Rhine, from 500 to 650 feet. On the Rhone it varied from three-quarters to one and one-half


12'6"

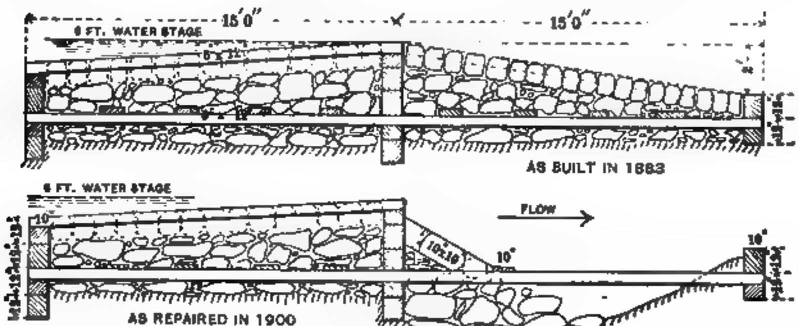

LOW WATER
ELEVATION
DIKES FROM LEFT BANK
NOTE:
ALL BRUSH IS UNDER LOW WATER. PILES ARE NOT USED WHERE ROCK BOTTOM IS CLOSE.
CHANNEL LINE

SECTIONS OF
CLOSING AND LONGITUDINAL DIKES
ON THE OHIO RIVER.

SCALE OF FEET
4 3 2 1 0 2 4 6




CLOSING DIKE AT WHEELING ISLAND, OHIO RIVER.



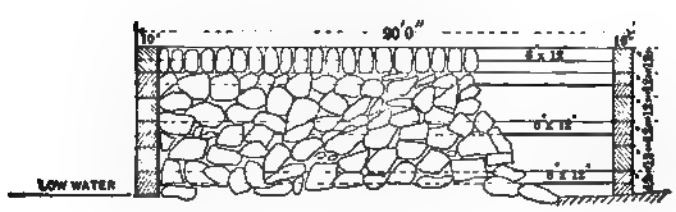

CLOSING DIKE AT BROWNS ISLAND, OHIO RIVER.

6 FT.

SECTION OF DIKE THROUGH CRIB

30'0"

OHIO RIVER.




BONANZA BAR DIKE, PORTSMOUTH, OHIO.
OHIO RIVER.

SECTIONS OF DIKES ON
FRENCH AND GERMAN RIVERS.
(From Rivières à Courant Libre and The Engineer)

SCALE OF FEET
10 0 10 20

times the channel width.* On the upper Mississippi, where more than 200 miles of spurs have been constructed (the dikes being usually run out from each bank opposite to one another), the rule of Schlichting has been used as a general basis, viz.: "Locate wing-dams or spurs running out from opposite sides so that where practicable their axes will intersect in mid-channel, and at the following distances apart; five-sevenths of the proposed channel width in straight reaches; one-half of the same width in concave reaches; and the full width in convex reaches."

The alignment or inclination of the spurs to the channel varies on different rivers from a slight inclination upstream to a similar inclination downstream. The German engineers have usually adopted the former, using an angle of 75 to 80 degrees, while in France examples of each type are found. On the Volga the inclination downstream was used. In Holland and in America the spurs are usually placed at 90 degrees with the channel, although cases are found with inclined spurs. On the upper Mississippi an inclination upstream has generally been used, the angle with the current being made 105 to 110 degrees in straight reaches, 100 to $102\frac{1}{2}$ degrees on concave banks, and 90 to 100 degrees on convex banks. On the Missouri straight and inclined spurs have each been employed. The practical effects of the various alignments are probably of slight importance, since there seems to be no marked difference in the results. We have been informed, however, by the Director of the Government experiment station in Berlin that when the heads of the spurs in their models were curved upstream, making a considerable angle with the stem of the dike, the eddies were found to be greatly reduced, since the obstruction to the flow became less sudden.

**Height of Dikes.**—The crests of the dikes, whether of the longitudinal or the spur type, should be theoretically at such a height that the contraction needed to produce the low-water depth of channel will obtain when the water has fallen to their level. If they are too low, the water will flow over them and pass to one side, only a part of it then flowing in the contracted channel, and the scouring will in consequence be too slow and the resulting depth insufficient. If they are too high an excess of water will be directed into the channel and the velocity may be such that the bottom will be scoured too much, in addition to the danger of erosion at the structure, due to its obstruction in higher stages.

The proper elevation is difficult to determine before construction, as it depends largely on local conditions and effects. On the Rhone the first dikes were built with their crests about 6 feet above low-water, while in later improvements and after experience had been gained this height was reduced to about 3 feet. In many of the longitudinal dikes the crest rose gradually to the point of maximum contraction and then sloped down. On the Rhine the tops of the spurs are about mean water level or average summer water level, and those of the longitudinal dikes are in many cases about 10 feet above low water. The spurs usually slope down from the bank with an inclination

* Annual Report, Chief of Engineers, U. S. A., 1902, p. 1777 and after. See also Plates 1 to 5, and Chapter V.

from 1 in 100 to 1 in 200, with the ends sloping 1 vertical to 4 horizontal (Fig. 72). The latter arrangement was found desirable in order to reduce the scour around the ends. On the Volga the height averaged $3\frac{1}{2}$ feet above low water. In America a height of 2 to 4 feet above ordinary low water has generally been adopted, that is, the dike rises 2 or 4 feet above the surface of the water in the improved channel. On the Missouri the spurs are usually built 2 feet above standard low water at their outer ends, and slope upwards till they reach the bank height near the shore, while on the upper Mississippi the crests are 4 feet above low water from St. Paul to Des Moines, and 6 feet from Des Moines to the Missouri. These heights were chosen as being the stage at which in early days the navigation of steamboats began to be uncertain. On the Ohio, where there are more than 100 dikes and closing dams, the heights range from one foot to 8 feet above low watei, with a general average of from 4 to 6 feet.

On the smaller streams it is sometimes necessary to modify the crest height after construction, raising it if the channel is found to shoal too much in high water or cutting it down if the scouring is found to be excessive when the water falls. Where the bed is of loose material it is often preferable to place the crest somewhat low until there has been opportunity to ascertain the height most suitable.

**Materials.**—In Europe the dikes are constructed almost entirely of riprap (usually in pieces from 80 to 120 pounds, or one-man size except the paving), as this material is permanent and adjusts itself to any minor scour, the pieces settling down as the erosion progresses. Where the action of the current is strong the entire dike is usually composed of this stone, but where practicable a core of gravel or other heavy material is used, faced with stone. Many such examples are to be found on the Rhone and the Rhine (Figs. 71 to 76), furnace slag having been used in many places on the latter river. In Russia piles, brush and similar temporary materials are largely used, owing to their cheapness compared with stone, and in America the same practice prevails except where rock is close at hand. On rivers where a dike of loose stone would be liable to injury by blows from ice during the spring floods, as on the Hudson, the stone is usually confined in a timber crib of smooth face, or extra heavy paving is used. In such cases there should be left as few openings as possible, as running ice tends to dislodge pieces that are laid irregularly or with too open joints. Dikes are also met with, as along city fronts, built of masonry or concrete. They have also been composed of gabions or baskets filled with gravel, small stone, or clay, as at certain places on the Volga, in India, in Japan, and elsewhere, but this method is unsatisfactory in strong currents, as the filling is soon washed away when the basket is injured or decays. On the upper Mississippi good results have been obtained with dikes of fascines or brush weighted with stone. On the Ohio about 80 per cent of the dikes are of riprap, 10 per cent of cribs filled with stone, and the remainder of brush and stone, or of piling and stone. Concrete has also been used in some of the closing dikes. Temporary dikes, such as are needed at times

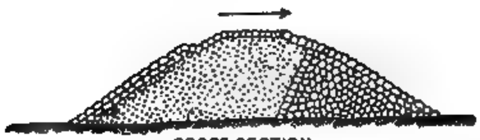



FIG. 71.


FIG. 72.

Spur-dikes as Built in 1880.



FIG. 73.—Cross-section of Spur-dikes as Built in 1885.

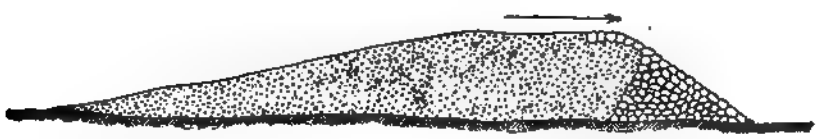


FIG. 74.—Cross-section of Gravel Spur-dikes as Built in 1882.

FIG. 75.—Cross-section of Training Dike Exposed to a Current on both Sides. (Golzheimer Island, 1887–1888.)

FIG. 76.—Cross-section of Training Dike Exposed to a Current on One Side only. (Mönchenwerth, 1884–1885.)

SECTIONS OF SPUR AND TRAINING DIKES ON THE RHINE.

for other construction, may be made of sacks filled with earth, of short plank driven into the bed, of bundles of brush tied with cord, etc.

**Construction of Longitudinal Dikes.**—Where built of riprap this type of dike usually consists of a pile of loose stone thrown into the water and allowed to take a natural slope. Along the river side additional stone is often needed at the toe, especially in sandy beds, as a protection against undermining, and if damage from ice may occur heavy close-set paving should be used above water. In some cases a trench is dredged into which the riprap is thrown. In India an apron is often placed in the dry bed, and when undermined by floods the stone falls down and protects the subaqueous slope, as described in Chapter V, under the heading "Protection on Indian Rivers." Where a core of other material is used a toe is first made below water and the dual construction of the upper portion is then proceeded with (Figs. 71 to 76). A close line of piles on each side filled

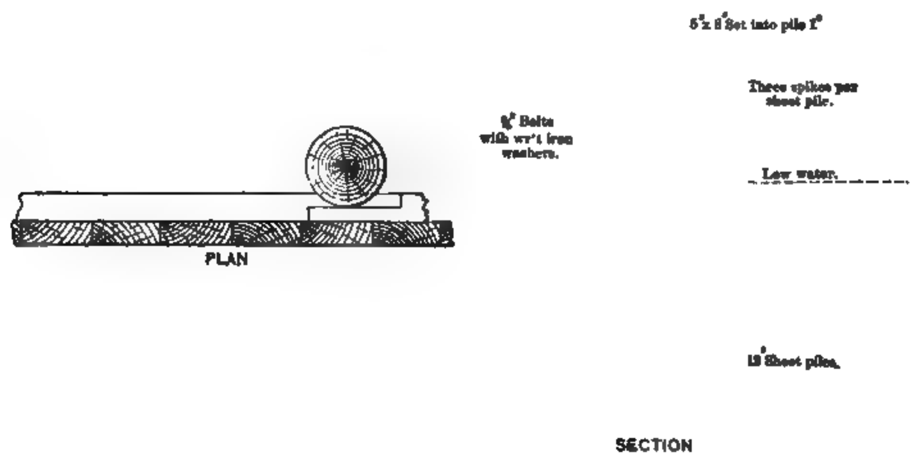



FIG. 77.—Sheet-pile Dike.

with stone, or cribwork similarly filled, has also been used. (See pages 147 and 148.) The cribs are usually built in place, but at some localities they have been built in quiet water and towed to the site. Where gravel filling is used a coping of heavy stone should be employed or the gravel will be washed out, and a heavy coping is also necessary with stone filling if ice is liable to do harm. This coping has in some cases been made of concrete. A cheap and effective method used on many streams consists in driving a line of piles 8 to 10 feet apart, closing the spaces between by a row of single plank, 2 or 3 inches thick, driven as sheet-piles (Fig. 77). These are usually driven close together, but good results may often be obtained by leaving a few inches between, allowing a small amount of water to pass through and causing sediment to be deposited behind the dike. We have seen excellent results follow from making the upstream portion of the dike continuous, so as to concentrate the flow, while the lower portion near and below the crest of the shoal was left as a row of piles 8 feet apart, giving the effect there of a permeable dike. The spaces behind rapidly silted up and the result was thus practically that of

a continuous dike. Another very cheap and effective type consists of two parallel rows of round piles, spaced 8 to 10 feet apart each way, the space between being filled with brush in pieces 10 feet or more in length (Fig. 78). The brush is then compacted by a pile hammer or by any means convenient, and a stout waling piece or tie rod is fastened across each bent of piles. This dike is at first semi-permeable and usually causes a rapid filling in behind it, although it is not quite as permanent as the plank dike and is more liable to be undermined, since the brush merely rests on the surface. The tendency to undermining will be reduced if the brush is well compacted during construction, as the pieces will tend to work downwards. After a short time the brush usually becomes filled with sediment. The cheaper forms of dike are more liable to injury from drift than the stronger types, but under suitable conditions they are usually less expensive in the long run.

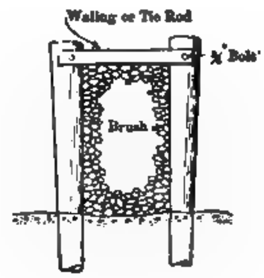


FIG. 78.—Plan and Section of Brush and Pile Dike.

In all forms of longitudinal dikes where placed on the concave side especial means must be taken, by a riprap apron or otherwise, to protect the junction with the bank, as this is the vulnerable point, the water tending to cut over and behind the end. Where on the convex side the danger is much less, but should also be guarded against.

The dike on the Loire shown by Fig. 68*a* (p. 143) was made of a single row of piles fitted with sliding wooden gates.

**Construction of Spur-dikes: Solid and Permeable.**—(See also "Closing Dikes," p. 158; and p. 164 and after.) Spur-dikes are usually constructed in accordance with the general methods just described, but while the scour on a longitudinal dike takes place along its face, the scour in a spur-dike takes place around the head, the stem of the dike being rarely endangered. This scour is much more pronounced with solid spurs than with permeable ones, and with swift currents a considerable protection of mattresses or riprap, or both, has been found necessary, extending above and below the head, the erosion always being worse on the downstream side. The permeable spur, however, by its slow checking of the current creates a minimum of scour and causes a filling out of the bank line. (See Fig. 68*a*, p. 143, and Figs. 87 and 88, p. 166; also Pl. 16 and others.) It is usually of slighter construction than the solid spur, and less expensive. This type is valueless, however, as a protection for caving banks such as those of the lower Mississippi, as it does not possess sufficient strength, nor could it withstand the currents in such places. It is used rather to build out bars, and on convex shores or in moderate currents. An example of its use is shown on Pl. 14, where the deposits shown were chiefly due to these structures, while at Elmot Bar (Pl. 17) they completely filled up in two years a channel originally 20 feet deep. (See also the Missouri dikes, Figs. 87 to 89, pp. 166 and 167.) It is

considered probable that after the dikes decay the deposits will in most cases still remain, while under water the brush will probably last for many years. These dikes are sometimes 2500 feet in length.

The permeable dike is of unknown antiquity, and has been used in India by the natives from time immemorial to improve the low-water channels of their sandy streams. Their method consists simply in driving bamboo stakes near together across the shallow channels they wish to close with the view of improving the main channel. The stakes are then tied together at the top and coarse grass mats are placed across them so as to hinder the passage of the water, and the results, while obliterated by the next flood, are said to be very effective.* Another simple and useful method which has been used in America, in India, in Australia and elsewhere, is the "Brownlow Weeds," consisting of brush fastened by the butts to a cord or wire and stretched across part of the shoal so as to cause a slackening of the current. The rope is fastened at intervals to stakes, or in narrow streams at each end only, and causes a sandbar to form below, thus concentrating the water in other channels. The usual present construction of permeable dikes is to drive a line of piles a few feet apart, extending about at right angles to the bank, between which walings and poles are fastened, making a kind of screen. In strong currents two or more rows of piles are sometimes needed, well braced together, as shown in Pl. 16. The poles or brush, while frequently damaged by drift, can be replaced at a small expense. Elsewhere, as on the Rhone, wicker mats supported by piles and walings have been used.

On the Missouri are to be found some permeable dikes with piles of concrete and with screens of brush. (See p. 169.)

On the Volga, where some spurs are to be found nearly two-thirds of a mile long, a fascine mattress foundation 2 feet thick is first placed extending from 7 to 26 feet upstream and from 26 to 39 feet downstream, of the proposed lines of the dike.† On this mattress is built up a superstructure of riprap or of fascines filled with small stone. These dikes are usually 7 feet wide on top with side slopes of 2 to 3, the crests being about $3\frac{1}{2}$ feet above low water. The foundation mattresses are usually built in winter upon the ice, and when ready the ice is cut out and removed and the mattresses lowered into place by ropes. The improvement of this river, however, is chiefly secured by dredging.

On the lower Mississippi, as described in Chapter V, the few spurs used are generally made of a cribwork of poles filled with riprap, although dikes of round piles have been used in the slower currents at New Orleans.

In Japan dikes are sometimes built up of bamboo baskets or crates woven like cylinders and filled with stone. The crates are first made in single lengths up to 50 feet or over, and run from 16 to 36 inches in diameter. After putting them in position the the riprap is forced in between the meshes, which are usually 4 or 5 inches across, until the crate is filled. The cylinders are placed alongside or across each other until the

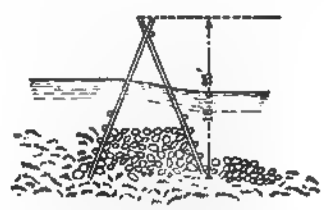
* "River Training and Control."

† Report on the Volga, Major F. A. Mahan, U. S. A.

required volume is obtained. They last two or three years, which is sometimes long enough to make permanent the desired changes in the river.

As mentioned on page 153 for longitudinal dikes, precaution must be taken at the bank end of all spurs to prevent the water from cutting around, especially when they are on the concave side.

**Spurs on the Ganges.**—An interesting type of spur has been employed on the Ganges, composed of crates or cribs filled with stone (Figs. 79 to 81), the cribs serving until they decay to keep the stone from being washed out. They have also been used as temporary dams to back up the water into irrigation canals. At the headworks of the Ganges Canal three of these dams were constructed annually, and served during the low-water season to hold up the pool. One had to be built in water 10 to 20 feet deep, and where the slope of the river was 10 feet per mile. To construct it a heavy rope was first passed across the river and propped up at intervals on groups of piles. The cribs were then brought out from shore one by one by a derrick boat which was attached by



CRIB ON BANK. FIG. 79. UNCOMPLETED SECTION OF DIKE. FIG. 80. COMPLETED SECTION OF DIKE. FIG. 81.

HURDLE DIKE AS USED IN INDIA.

block and tackle to the rope, and were lowered into place, being weighted with stone as they settled. Each crib was lashed to its neighbor, and the whole was gradually and uniformly filled with stone and made tight with bundles of brush or grass, backed with gravel and small stone. (Figs. 80 and 81.)* An example of masonry spurs used in this locality after 1879 is shown on Pl. 13.

**Submerged Spurs or Ground-sills.**—We have seen that one of the effects of contracting a river is the scouring out of its bed, and that this action may not always cease at the point desired, and may in consequence lead to undesirable complications. To avoid this it was proposed many years ago to adopt in connection with the dikes a system of low dams or sills placed at intervals across or partly across the bed. They were to coincide approximately with the surface and in reality form immovable bars. This general idea has been applied to the Elbe, Rhine, Rhone, and other rivers, the works being called submerged spurs, ground-sills, or "Grundschwellen." It will be noted that they are not considered as a complete system of improvement, but are employed in connection with regulating and contracting dikes. The sills may be used either to

* "Training Works of the Ganges at Hardwar," W. Ward-Smith, C.E.

hold the river-bed, or to raise it at points where too great depths exist. They are built of riprap in nearly the same way as the spur-dikes, and usually start from or below low-water mark, inclined either at 90 degrees with the channel or in a direction upstream at an angle varying between 60 and 80 degrees. The top slope as it goes out into the river is about 1 in 10 to 15 for a short distance and then decreases considerably. (Fig. 82, p. 159.)

The earliest example of submerged spurs is found on the Ruhr, a little river which falls into the Rhine at Ruhrort, and to which the coal mines of Westphalia gave an unusual importance in the system of transportation routes prior to the construction of railroads. The Ruhr was made navigable for a length of $46\frac{1}{2}$ miles by a canalization comprising eleven locks and dams, supplemented at the upper ends of the pools by dikes. The depth at the head of one of these pools became reduced by too much scour, so that the slope in low water disappeared and the miter-sill of the upper lock was uncovered, stopping navigation. The engineers restored the slope by constructing throughout the pool a series of submerged transverse dikes, which divided the total fall so as to restore the depths necessary for navigation during low water.

Great use was made of submerged spurs in the improvement of the Elbe, and the German engineers were naturally led to adopt them by the system of works chosen. The aim was to improve the channel by contraction, so as to create in the natural bed a minor bed whose width, after taking into account the resistance of the bottom, should be suited to the conditions of slope and of discharge. Instead, however, of controlling this minor bed by longitudinal dikes, they have usually created it by building spurs which extend into the stream from each bank, and terminate on the line adopted for the desired bank of the minor bed. (See Pls. 3, 4 and 5, and Chapter II.) As might have been expected, the heads of the spurs were usually attacked by the current, and scour was produced, threatening the existence of the spurs, and destroying the regularity of the channel. The engineers were thus led to prolong their spurs under water, advancing into the bed of the river in order to protect them and limit the effect of the scour.

The works of regulation on the Elbe were as follows:

First, next to the bank was the spur-dike intended to contract the natural bed. Usually this dike at its root on the bank was at a height of $8\frac{1}{2}$ feet above low water, and at its outer end at $6\frac{1}{4}$ feet above low water.

Second, in prolongation of this was the submerged spur or sill which limited the scour caused by the dike proper and protected its head. The starting-point of the sill was about 5 feet below low water, and the top had a slope which was from 1 foot in 25 to 1 foot in 12.

The works thus placed are said to have completely answered the expectation of the engineers. The alluvial deposit has filled up the places that had been scoured out, and has permitted a reclamation of the spaces between the spurs. The works have thus become protected naturally, and other results have been produced which are still more important

for navigation by diverting the current from the heads of the spurs, and pushing it forward into the open channel towards the line of greatest depth. Barges and rafts floating with the current, and fleets of boats in tow have ceased to be carried against the heads of these spurs, and are now kept by the natural forces in the middle of the channel, or at least at a sufficient distance from the banks. The submerged sills have thus caused the disappearance of one of the inconveniences which could be urged against the system of contraction by spurs—that of forming obstructions dangerous to navigation. The improvement was so marked that sharp bends, formerly difficult to pass, became free from danger.

The sills are usually short, but where much scour had been produced, or was to be feared, they were constructed with a view to the regulation of the bottom, and consequently of the slope, rather than to protect the spur-dikes. Hence the works on the Elbe made the depths in a great measure uniform. Formerly this river, as with most streams, had a series of pools, more or less deep, and separated by bars, but after improvement the bottom had a nearly uniform depth. It is true that the velocity of the current was increased in the pools, but the advantage of a regular depth of water over the whole route was such that even this inconvenience was not of great consequence.

The work of regulation by spur-dikes in Germany was not completed immediately, as was often the case when longitudinal dikes were built. Their construction was governed by methods and rules which permitted a considerable liberty of action on the part of the engineer. Hence, when an improvement might be liable to lead to undue displacement of the bed, especially in the concave parts of the stream, the dikes were commenced slowly, and were stopped at a provisional curve. Time was then taken to watch the effect produced. If the action of the current attacked the bottom at the outer ends of the spurs, they were prolonged by sills which were to form a foundation for the succeeding part of the dike, and which in the meantime would immediately stop the scour and tend to cause deposits. This gradual construction, or experimenting with the dikes, was done not only as regards their length, but also as regards their height, and when a certain depth had been obtained, submerged works were commenced, which at a later period were built higher if necessary. Remarkable amounts of deposit were thus obtained in many cases, and dikes which could only have been constructed at great cost if in deep water were built gradually, and finished on a bottom that had been raised without difficulty and at a small cost. Such results on convex banks were very rapid, but on concave banks the system gave less satisfactory results. Scour at the heads of the dikes always took place, and could not be checked except by the precautions and the methods of gradual construction just described. It was doubtless this fact which induced the German engineers to adopt and generalize in so remarkable a manner the use of submerged spurs, the other advantages of which could not have been discovered except by the experience acquired after their construction. In fact, in all that portion of the Elbe which was under the control of Prussian engineers, and where they applied sills,

the two banks showed in time a remarkable regularization, and one which constantly improved, so that the indentations visible between the successive dikes gradually filled up and new and regular banks grew into existence.

It will be seen from what has been said that regulation of the bed as well as of the banks was considered necessary in Germany, and that the engineers not only fixed by a series of dikes the width of the low-water bed, but also secured this channel against scour by means of ground-sills. The result was that the bed rose to the level of these sills and assumed a regular slope, and the banks became filled out by deposits between the spur-dikes. The advantages of this system are summed up by M. Jacquet as follows:

The nearly uniform distribution of the slope, and the consequent disappearance of the bars over which the depth of water was not in harmony with the general regimen of the river.

The protection of the works of regulation, and in general of all the works that were attacked by shore currents.

The removal or transfer of the line of greatest depth and greatest velocity to a certain distance in front of the shore works, and the consequent suppression of the dangers which dikes might offer to descending navigation.

The regulation of depths and velocities in the same cross-section.

The creation of a uniform depth of channel throughout the length of river subject to the same regimen, and sometimes throughout the whole course of a river, as happened on the Elbe, so that boats can everywhere find nearly the same depth of water.

Submerged sills, however, do not always produce a leveling or filling up of the bed, especially where used to reduce excessive depths, as in these cases they appear to act as submerged dams, causing vertical eddies which prevent local deposits. They have been rarely used in America as yet.

These sills have also been employed with success in preventing or correcting the inconveniences resulting from the construction of a bridge. Thus, on the Saône at Lyons, the bridge of Ainay, the arches of which were narrow and obstructed with offsets of masonry, never afforded in high water more than one arch practicable for navigation, and that one could be used only with difficulty because of the swift current. The later construction of submerged sills below sufficiently distributed the fall so that boats were able without serious difficulty to pass the arches which they had not been able to approach before.

The spacing of submerged sills, as with other forms of dikes, is dependent on local conditions. On the Rhone they are stated to have been placed at distances from two-thirds to nine-tenths, and on the Rhine from one-third to two-thirds, of the channel width. Fig. 82 shows a type of sill used in the latter river.

**Closing Dikes.**—Rivers often divide into two or more passages. The effect of this is to distribute the flow, and in order to create in the stream conditions more favorable to navigation at low-water stages it may be necessary to close the secondary arms.

This is generally accomplished by means of low dams or dikes across these arms, several being sometimes required in a single channel. The construction of these dikes presents some special difficulties, not encountered in dikes along a shore. Where built of riprap the usual method is as follows: A layer of riprap is placed across the site of the proposed work, extending sufficiently dowstream to act as an apron to receive and carry off the water overflowing the dike, and for a sufficient distance upstream to form the floor of the structure. A rule given by De Mas for the width of this bed of stone is that it shall be fifteen times the fall produced. Its thickness must be determined with reference to the character of bed upon which it is placed, for when the bottom is soft it is liable to sink. Upon the bed thus prepared the main structure is built, in layers extending its entire length. The upstream slope is about 3 of base to 2 of height, while that of the lower side is generally less, being about 2 to 1 for low dikes, and decreasing as they become higher. As the construction proceeds the water will rise over the structure and produce settlement at various points, or possibly along the whole dike. Additional material must

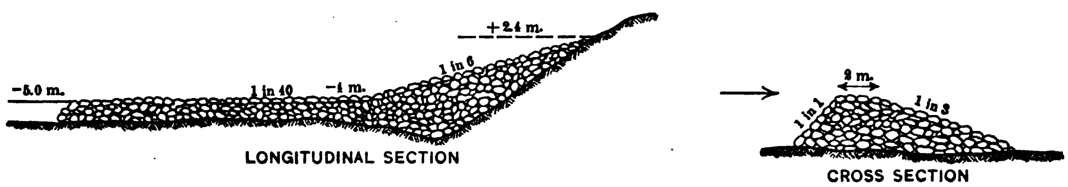



FIG. 82—Submerged Sill.

at once be put in in order that the profile may be maintained at the desired elevation; otherwise a loss of some of the material already in position may follow through undermining. It is, therefore, advisable to proceed with as great rapidity as practicable. The paving or coping should be of selected stones, hand-placed and roughly jointed, in order that the surface presented to the overflow may be uniform and solid. Piles can be used to good advantage in a foundation for this character of structure, and may effect a saving in stone.

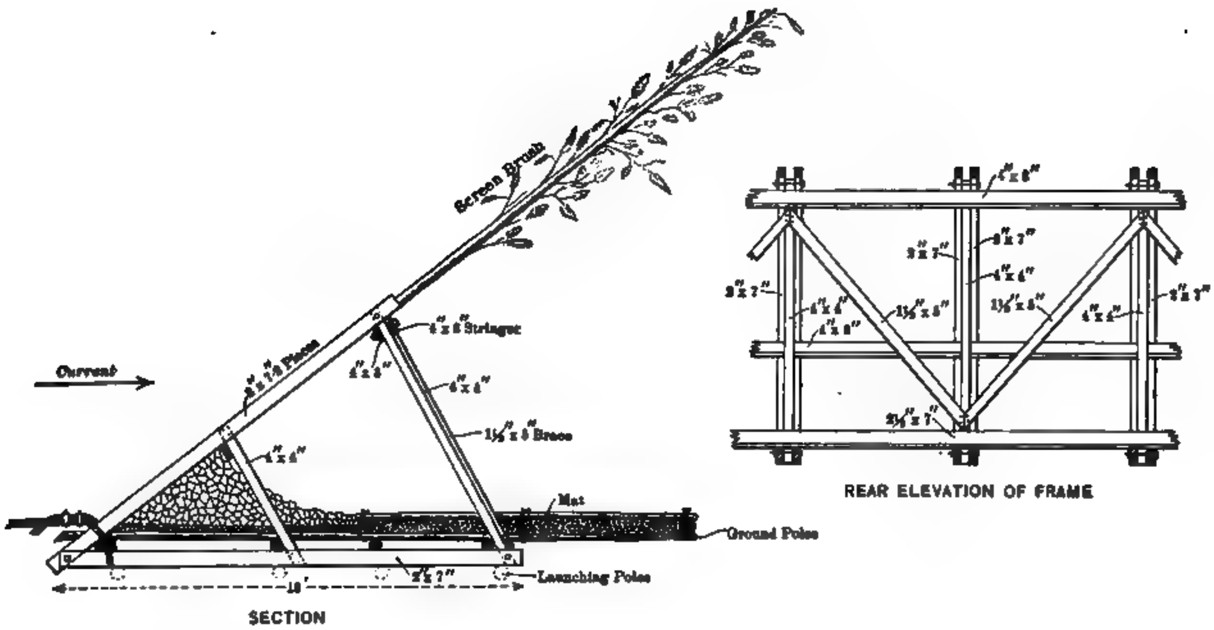
Where the channel is to be closed by timber cribs filled with stone, the general method of construction is the same as described for the building of fixed dams in Part II, and special care must be taken to prevent the water from cutting under or around the structure. The elaborateness of the precautions will of course depend on the nature of the banks and bed, and the volume of water to be controlled, but the dangers from lack of sufficient protection may be considerable. Thus on the Wabash River, one of the large tributaries of the Ohio, a crib dike was built many years ago to close a cut-off which had shortened the river's course several miles. Owing to complications with a landowner only a small amount of protection could be placed at one abutment, and when a flood came this weak spot was attacked, extensive surface erosion began, and the dam was eventually flanked, the river cutting a new channel many hundreds of feet in width around the end of the

structure. The cribwork was eventually buried by deposits and covered with a growth of shrubs and trees until hardly a trace of it could be seen.

The impermeable type of closing dike produces by its obstruction to the flow dangers of undermining or of erosion, as in the example just quoted, but the permeable type tends to accomplish its purpose more easily and cheaply and with a minimum danger. This type is usually made of brush and stone or of brush and piles, as described for spur-dikes (p. 153) and has been employed to a considerable extent in America. By its causing the flow to slacken, deposits of sediment occur, and thus the river is made to close the channel itself. In several instances where a tight closing dike had been destroyed by undermining, a permeable one has been built in its place with success. The construction should proceed gradually so that the river may have time to adjust its other channels to take the increased flow of water, as a too sudden obstruction may lead to unlooked for changes and additional expense both along the banks and in the bed. Fig. 83 shows a permeable dike employed on the lower Mississippi and on the Missouri as a means of closing side channels and building up bars, and which has given very satisfactory results. Fig. 84 shows a type much used on the upper Mississippi, and a description of a large one built on the lower Mississippi will be found on page 175. Types used on the Ohio are shown on page 148.

The type shown in Fig. 83 was first tried on the lower Mississippi in 1898. The sections are built on barges in 250-foot lengths and launched like mattresses. Anchor piles, 50 feet apart, are used to hold the sections in the water, wire ropes being fastened near their bases, to which the sections are secured when ready for sinking. The piles are sawed half through near the bottom before driving, and when done with they are pushed over by a steamboat and broken, leaving the river clear. A section dives down perpendicularly when launched but quickly rights itself, and as soon as it is settled scour begins on the upstream side; this stops after twenty-four to thirty-six hours. Sometimes the scour reaches a depth of 4 feet, and there is always erosion around the end. To prevent inconvenient though temporary scour there, the ground poles project several feet beyond the end, and on these the next section rests, thus forming a kind of mattress protection. At the ends a small fascine mattress is generally used, and the bank end must always be revetted or the river will cut round it.

Experience on the lower Mississippi has indicated that in closing chutes, little head should be created, because of undermining, and that it may be advisable to use two or more works, and thus divide the head. They should be put toward the middle of the chute, and it is usually best to use some kind of permeable dike so as to check the current slowly. The ends must in all cases be protected, and drift should be allowed free passage, or it may gorge and cause washouts. Above all, the deposits should be created slowly or the congestion of the currents elsewhere may cause increased erosion and undesired changes in the flow. Much assistance in the closing of a side channel is often derived from the growth of willows which springs




SKETCH
SHOWING DETAILS OF
TRIANGULAR FRAME DIKE
AS USED ON THE MISSOURI,
& LOWER MISSISSIPPI RIVERS.
SCALE OF FEET

TOP VIEW OF FRAMES

TOP VIEW OF STONE BALLAST AND MAT

TOP VIEW OF DIKE COMPLETE

Fig. 83.

up on the new deposits during the low-water season and acts as a permeable dike of steadily increasing influence.

**Experiments with Models.**—Between 1895 and 1900 there was established at the University of Dresden by Professor Engels a laboratory in which portions of a river could be represented by models, and the effects of various arrangements of dikes traced. The scale, velocity, and amount of flow and character of the materials of the bed were all represented in suitable proportion, and experiments could thus be conducted to indicate the probable effects of an improvement. In 1902 the German Government established a similar station at Berlin where investigations are made prior to actual construction, and while the agreement of the results given by a model with those actually worked out by the river cannot be compared until after a lapse of several years, indications point to the method being of invaluable assistance. During a visit to this station we were

CROSS-SECTION OF CLOSING DAM
AS USED ON THE UPPER PORTION OF THE MISSISSIPPI
PROPORTION OF ROCK & BRUSH 1:1

FIG. 84.

shown an experiment under way for improving a shallow bend in the Wartha by means of spurs from each bank. The scale of the model, which was about 20 feet long, was 1 to 100, both vertical and horizontal, and the size of sand composing the bed was about $\frac{1}{8}$ of that found in the river at the site in question. This size, together with the proportionate velocity of water, was determined by formula. The dikes were filled up level from crest to crest with this sand so as to obliterate the eddies, shot being used to represent the riprap which would be employed for protection in the actual construction. The water was regulated to represent all phases of a season's flow, both low and high water being given their proportionate durations and amounts. This experiment, which had been under way with the same arrangement of dikes for some weeks, and was to continue for some weeks longer, had indicated that the low-water depth could be increased from about 3 feet to about $4\frac{1}{2}$ feet.

**Experience with Dikes in America.** *Arkansas River.*—The following is a general summary of the experience obtained on the Arkansas River during a period of about twenty years.* The bed and banks of this stream are of a shifting, sandy character, and the floods often come swiftly and are of considerable height.

The first attempt at a general improvement was made with permeable pile dikes, consisting of two rows of piles about 8 feet apart, with brush woven between and protected above and below by light mattresses, as shown on Pl. 15. These were designed to concentrate the flow when the stage was 7 feet above low water. They were effective when used at the lower end of a bar for the purpose of making it build downstream, but when used at the upper end, or placed for the purpose of deflecting a current, the brush proved too weak, and it was soon found that the drift worked serious injury to them. They were in consequence changed to pile dikes filled with stone, and these proved much more satisfactory. It was necessary, however, to refill them every few years as the stone settled into the sandy bed.† Additional protection had also to be placed where the dike ended in the bank, the method eventually adopted being to riprap the bank for 50 feet above and 100 feet below the dike. The bank opposite to the dike was in many cases eroded by the deflection of the current, and had to be protected by the ordinary methods of mattresses and riprap, the mattresses varying in width from 60 feet in bends of 6 to 10 feet depth to 120 feet in bends 20 feet in depth. The bank above the edge of the mattress, which stopped at low-water level, was graded and protected with riprap up to flood level. It was found in all cases, both with the dikes and with shore protection, that unless the adjacent bank was protected up to that level the water would cut out the soil along the upper edge and sometimes cause much injury to the structure.

After some years' experience with pile-and-stone dikes, the type was gradually changed to stone dikes composed entirely of mattress-and-stone foundations with riprap above. (See Pl. 15.) In building these the shore protections were first constructed of material and dimensions as described and shown for the pile-and-stone dikes. Very little settlement occurred with these dikes, but where the sides exposed to the current and to drift consisted of stones of ordinary or one-man size, some damage resulted from displacement. (For costs, see p. 178.)

*Cape Fear River, North Carolina.*—On this river, which possesses alluvial banks and a sandy bed, the best results were obtained with permeable pile dikes, consisting of two rows of piles with brush filling, rising to about 4 feet above low water. They were generally built as longitudinal dikes, or as spurs set at a considerable angle downstream, and were found to produce a good low-water channel, and to be reasonably permanent. Stone spur-dikes were also used, but were much less satisfactory, as it was found that in high water the current excavated the bed on their downstream side, while with the permeable pile-and-brush dikes the bed would fill instead. On one stretch 43 miles

* Annual Report, Chief of Engineers, 1902, p. 1576.

† This subsidence almost invariably takes place where a river bed is of sand.

long the aggregate length of the shoals was 17 miles, all of which was improved by dikes. In the unimproved condition the depth fell in summer to about 2 feet and navigation lasted only about four months in the year, while after the improvement the minimum depth was about $2\frac{3}{4}$ feet, and navigation could be carried on the greater portion of the year.

*Missouri River.*—The characteristics of this river and an outline of the work done on it have been described in the Chapter on Regulation (p. 89 and after). The system of permeable pile dikes has been brought to an unusual development during an experience of many years, and the possibility demonstrated of successfully controlling this difficult stream. Thus at Atchison about the year 1908 the river had shifted its channel at a railway bridge so that the opening under the drawspan became too shallow for boats. It was trained back by the gradual effect of a permeable dike built across the new channel; the latter in a few months had silted up and the river had moved back to its original channel.

A description of the structures now used as standard dikes is given below, and a description of the experiments with "bank-heads" will be found in Chapter V.*

"Permeable dikes are employed where it is desired to contract the river or to force it to any desired position, or to fair out the shore line, advantage being taken of the fact that in high water the river carries in suspension a large quantity of sediment which it deposits at points where the current is checked. These dikes consist essentially of, first, a system of piling driven in rows a short distance apart and braced to resist the action of the water or of ice and drift; second, a woven willow foot mattress to prevent scour, and, third, screening extending from the top grade line of the dike to the bottom of the river, to cut off a portion of the flow of water through the dike, thus causing the current to slow up and deposit a portion of the sediment carried in suspension. The dikes are run out level with the top of the bank, or 2 feet above standard high water, to near the standard high-water contour of the proposed rectified shore and from there sloped down to as near 2 feet above standard low water as the stage of river at the time permits.

"When located on the bar side of the river, or the convex side of bends, the exposure is generally or at least frequently such that the dike is made of increasing strength from the bank out; i.e., commencing at the bank with 1-row work and changing successively to 2-row, 3-row, and 4-row work as the exposure increases with distance.

"When located on the concave side of bends the dikes, on account of the exposure, are seldom of lighter construction—fewer rows of piling—at their bank ends than at their stream or outer ends, and in some instances the heaviest construction is near the bank. This is found necessary where the channel lies close to the bank and pile penetrations

* A history of the methods used and results attained on the Missouri is given in the Transactions Am. Soc. C.E., June, 1905, by S. Waters Fox, M. Am. Soc. C.E., by whose courtesy the accompanying matter and cuts are reproduced.

are limited by excessive depth of water, impenetrable bottom, or other causes. The lower dikes of a group are made of lighter construction than the one farthest upstream, excepting that portion of each which projects beyond the influence of the dike next above

FIG. 85.—Dike 1 B, Practically Completed.

FIG. 86.—T-dike, in Wilhoite Bend.

DIKES ON THE MISSOURI RIVER.

it.* The piles are driven in rows 10 feet apart from center to center, being about 10 feet apart in the rows. Additional piles are driven at the outer end as shown. The axis

* The construction of the three-row dike, which is the type in most general use on the river, is shown on Pl. 16, and cuts of this and other structures on Figs. 85 to 89.

of the dike is perpendicular to or inclined to the bank, as local conditions may require. Yellow pine, oak, and cottonwood piles, not less than 14 inches or more than 19 inches in diameter at the butt and not less than 8½ inches in diameter at the point, are used, and they are driven to a penetration of 25 feet.

FIG. 87.—Little Tavern Dikes, Nearing Completion.

FIG. 88.—Accretions Due to Little Tavern Dikes, as They Appeared Two Years After Completion of the Dikes.

DIKES ON THE MISSOURI RIVER.

"The foot mattress is woven from the shore to the outer end of the dike, either before the piles are driven or immediately afterwards, before the bracing is put on. The mattress for a 3-row dike is 60 feet in width, extending 25 feet above the dike and 15 feet below, and is woven and strengthened with wire strands. At the shore end it is extended

upstream 20 feet and at the outer end it is widened to 95 feet. When the dike piles are driven before the mattress is woven it is necessary to use, in addition to the barge above the dike, a small barge below and small punts between the rows of piling. Anchor piles are driven at intervals about 60 to 80 feet above the upper end of the mattress, and cables from these hold the mattress in position during construction. After the mattress is completed it is loaded with rock to an average thickness of 3 inches and sunk to the bottom, the upper 10 or 15 feet being more heavily loaded than the remainder, to prevent the rolling of the upper edge by the force of the current.

"The system of bracing used is indicated on Pl. 16. The upper end of one of each pair of diagonals is extended so as to protect the end of the other, and extra bracing

FIG. 89.—Method Used in Building a Continuous Abattis Over Water. Looking Inshore.

DIKES ON THE MISSOURI RIVER.

is put on at the outer end of the dike. Bolts $\frac{3}{4}$ of an inch in diameter are used for bolting the bracing to the piles.

"After the dike is braced the screening poles are put on, the lower ends being trimmed to a point and pushed well through the foot mattress and the upper ends being nailed to the upper and lower waling pieces of the middle row of piles. The screening poles are from $1\frac{1}{4}$ to $2\frac{1}{2}$ inches in diameter at the top and from 2 to 4 inches in diameter at the butts. A pile supported at one end by the middle pile of the inner bent and at the other by a pile head planted at the top of the graded bank is placed, in order that the screening may be continued to the top of the bank. The screening poles are spaced so that about one-half of the section is cut off at the inner end of the dike and about one-quarter at the outer end, the spacing varying as nearly uniformly as possible between

these points. At the shore end of the dikes the bank is graded to a slope of 1 to 2 and revetted to the top of the bank, or 2 feet above standard high water.

"In cases where circumstances prevent the procuring or use of poles for screen work wire netting is placed in front of the upper line of piling. Area of mesh, about 24 square inches.

"After the outline of the accretions becomes defined, or within a period of three years after the completion of the dike, that portion of the dike beyond the accretions is reenforced by filling in with mattress and stone, to form a submerged spur extending on an approximately uniform slope about 40 feet beyond the dike head.

"All double-direct braces are provided with filling blocks 30 inches long, fitted close up to the pile at each end of the brace, and held in place by two ¾-inch screw bolts. The object of this device is to relieve or reenforce the bolts which fasten the brace to the pile.

"The use of wire strand ties* to transmit stress from the top of dike in the lower row to the base of the structure at the upper row as a measure of stability previously obtained by the more expensive method of double-system bracing or additional row of piles. The ties are of several parts of ⅜-inch strand, or one or more parts of ¾-inch strand as indicated by the stress. They are usually attached to the upper pile before driving, at a point that when driven will be on or near bottom, a round turn being taken on the pile and the bight of it fastened there by a staple; if a single-part tie, the short end is clipped on the other close up to the pile, and the tie, whether of one or more parts, is then lashed up alongside of the pile until the mattress shall have been sunk in place, when it is made fast at the top of the pile in the lower row.

"There are two things which mar the efficiency of pile cross-dikes:

"First.—Immense quantities of drift-wood, borne on the surface and at all depths during flood stages, find lodgement to a greater or less extent in unevenly distributed masses upon the structures. This often results in breaching the dike, by overturning it, or, by crushing the structural parts; scour, introduced by concentrated flow, or excessive head, may be a primary, or, contributory cause, and, of course, once a breach is formed it rapidly widens, due to scour, until head has been dissipated to such an extent that velocities are reduced below that at which scour can take place. Even though the structure be not breached, its curtain may be rendered practically inoperative by drift-wood, so that the flow through the dike is very uneven and the deposits formed are correspondingly irregular.

"Second.—There is always more or less of a pothole or trench at the stream end of the dike, which attracts the flow and prevents the formation or maintenance of deposits quite out to the end of the structure; more or less eddy action is in persistent attendance, and, as the structure deteriorates with age, constant exposure to the forces of the river

* Not shown on Pl. 16.

is more and more likely to destroy it; and, once the outer end of a dike is destroyed, the remaining portion yields more readily. Scour at the stream ends of dikes is first caused by increased velocity due to release of head on the structure. The pothole or trench is formed and maintained as the result of scour in the presence of a fixed object of limited extent; and the form, area and depth of the pothole depend upon the form and extent of the structure, the head due to the resistance it offers to flow, and the character of the river-bed at that place.

"Initial scour having occurred, flow is attracted, and, because of the fixed object (the outer end of the dike), velocity and, therefore, scouring capacity are increased. Increase in depth and velocity is followed by decrease of local width. A bar, or reef, over which the attracted flow pours into the deepened section, moves down toward the structure, increasing the scouring effect and continuing to advance until its lower face is swept by a current strong enough to carry away, as fast as contributed, the material brought in over its crest. The pothole or trench thus formed is defined on one side by the bar, and on the other by the structure, and it conforms somewhat to the latter. The trench extends but a short distance beyond the structure, being most pronounced in front of it, or slightly below the point of strongest impact against the structure. Easement of flow is found immediately below the dike, and the slackened current being unable to carry the materials scoured out from, or brought through, the trench, drops them, forming a bar; and the tendency is for the flow to divide on the bar, forming two waterways. The location, height, and extent of the bar, and the predominance of one or the other of the two waterways depend largely upon the form and size of the structure, but vary also with stage, approach, and other elements of flow.

"It will be seen, therefore, that in reaches of river requiring advancement of both banks, the use of cross-dikes will result in a more or less divided flow; the cross-section of a reach so treated will be characterized by shallow mid-stream depths and relatively greater depths along both banks. In straight reaches, the flow may approximate an equal division along each bank; in curved reaches, up to the limits of curvature when chording effect is operative, the flow would be increasingly on the concave side. Several devices have been tried with a view to remedying or ameliorating these effects, but with only a small measure of success.

"Though no longitudinal dikes, with or without stems, were built in the stretch of systematically improved river in First Reach, the results obtained by them, elsewhere in the river, are thought to be of importance and promise.

"The first longitudinal pile dike on the Missouri River was built by the commission in 1895–96 for the purpose of masking a pronounced bay in the shore line of the left bank above the Interstate bridge, above Omaha, Nebraska. As far as completed, it was a marked success, promptly accomplishing and maintaining, without repair or additional attention of any kind, the object for which it was built. It conformed to the proposed line of rectified shore, and connected with the outer ends of a series of cross-

dikes built from the main bank on lines about normal to the flow. Its length was 2600 feet."

During 1909–1910 two permeable dikes were built on the Republican River, at Fort Riley, Kansas, in which the piles and bracing were made of concrete, and only the screens were of non-permanent material, the customary wooden poles being used therefor. The dikes were of the 2-row type (see Fig. 90), one being 400 feet and one 800 feet in length (extended later to 1100 feet). The piles were made 11 inches square and 30 feet long, reinforced with 4 longitudinal rods of $\frac{3}{4}$-inch diameter and with $\frac{1}{2}$-inch diameter rings spaced 2 feet centers. The braces were $8 \times 8$ inches, reinforced with 4 rods of $\frac{3}{8}$-inch diameter. The concrete was of a 1-2-4 mixture.

Details of costs of the Missouri dikes will be found on p. 177.

*Mississippi River.*—A résumé of the characteristics and of the methods employed for the improvement of this river will be found in Chapters I to III and V. The upper

SECTION OF SPUR DIKE, UPPER MISSISSIPPI RIVER

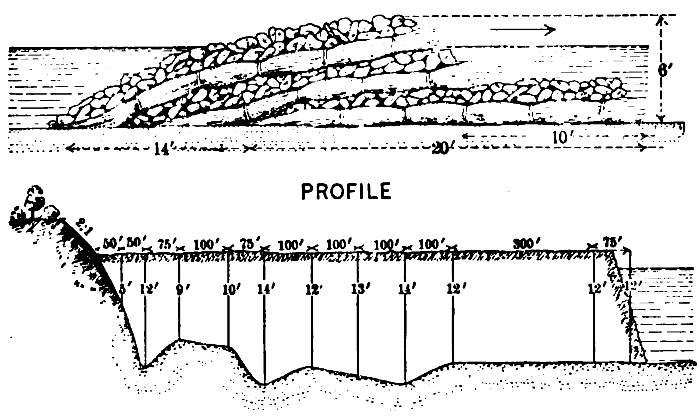



FIG. 91.

portion, extending from St. Paul to the mouth of the Missouri, has been improved principally by dikes and revetment; the middle portion, from the Missouri to the mouth of the Ohio, by dikes, revetment and dredging; and the lower portion, by revetment and dredging. In the upper portion, which is of moderate slope, the system of diking consists almost entirely of spurs, one set being usually placed on each bank and about opposite to each other, so that both sides of the river are contracted. (See Pl. 6.) These spurs are built of rock and of brush fascines, as shown in Figs. 91 and 92, and have proved in general very satisfactory. Details as to spacing, etc., have been given on p. 149, and details of cost will be found on p. 178. The usual erosion around the ends of the dikes takes place after construction, but where not pronounced the hole often becomes partly filled as the channel reaches equilibrium. In some cases, however, the end of the spur has been undermined, and additional protection of rock and brush has had to be used. Longitudinal dikes have been employed very little in this portion of the river, except in special cases as at landings or to prolong the lower end of an island. It was

FIG. 90.—Permeable Dike with Pole Screen, and Concrete Piles and Braces. Constructed in 1910 on the Republican River, at Fort Riley, Kansas.

FIG. 92.—Models of Brush and Rock Dams and Dikes. Improvement of the Upper Mississippi River.

found that the spurs offered much better facilities for the deposit of the sand while the river was scouring its new channel, as such deposit rarely took place with longitudinal dikes. The increase of experience with this improvement has led to the adoption in many places of spurs on one bank only, revetment being used on the other. This was found preferable for boats, as navigation was much easier when the channel passed from bank to bank than when it was trained to keep in the middle of a wide river.

In the contraction of the bed it was found that the sand was moved to an extent which caused the shoaling of many previously good channels below, the pools not being deep enough to hold the material scoured away. (See also p. 71.) Dredging to assist the work of the river could not be employed to any extent during the earlier improvement, so that all surplus material had to be disposed of by the river itself.

The works used between the mouths of the Missouri and Ohio Rivers, where a channel width of about 2500 feet was desired, consisted at first of spur-dikes of riprap. As these did not prove altogether satisfactory, especially in failing to create deposits, a permeable or hurdle type was substituted in 1879, and has been used since. It consists of one or more rows of piles, driven in clusters of three, with a foundation mattress extending about 35 feet upstream and from 40 to 60 feet downstream. The piles are driven through the mattress, and a curtain of brush or poles is usually placed between the clusters. The channel end is protected by a T-head of piles extending 50 feet above and 100 feet below the dike, with a foundation mattress 120 feet wide by 300 feet long, and the shore end is revetted for about 300 feet. After the deposits have reached a height of 16 to 18 feet above low water, a growth of willows springs up and assists in creating further deposits. These dikes are placed from 1000 to 1500 feet apart.

Temporary dikes, to assist in local scour, have also been used. They consist of a row of piles and stringers supporting sheets of corrugated iron 10 feet wide and 10 to 20 feet high. A fascine mattress 8 to 10 feet wide, weighted with stone, is used to guard against scour along the toe, and at the end of the low-water season the piles and sheets are taken up for future use.*

The improvement of the lower river is carried out by revetment and dredging, and is affected indirectly by the levees. When work was first undertaken, an experiment was begun to determine whether this portion of the stream was susceptible to improvement by dikes, as from the mouth of the Ohio to Baton Rouge it is characterized by swift and deep currents and much erosion. The results, while generally satisfactory, were slow and expensive, and the principle was therefore abandoned in favor of regulation by revetment and dredging. The experiment was unique in the annals of engineering, owing to the great size and pronounced characteristics of the river, and its history is briefly summarized herewith.

The portion selected was Plum Point Reach, about 160 miles below the mouth of the Ohio. This reach was one of the worst on the river, and was almost 20 miles long,

* Temporary dikes have also been used on the Loire and elsewhere.

of which 10 miles lay nearly straight, and was encumbered by many small shifting sandbars and two large and permanent ones. The width was 1½ miles in many places at low water and 2 miles in floods. The general plan of the works, modified later by the addition of some cross-dikes, is shown on Pl. 17, the object being to contract the flow at medium and low stages by the use of permeable longitudinal dikes, and to protect the bank where necessary by revetment. All the dikes were of piles, and permeable. The first type used is shown in Fig. *A*, Pl. 18, and was built in 1881 of bents of piles, and was placed at the upper end of Bullerton Bar. It caused much deposit below by the slight checking of the current due to the wire netting screen, but it was too weak and had no foot mat, and was washed out by the first flood. In the following year another dike, of design as shown in Fig. *B*, was commenced at the upper end of upper Osceola Bar, but before completion a rise came which scoured out many bents of piles, while others were destroyed by drift, practically wrecking the structure. Later in the year it was rebuilt and other dikes begun, the type of Fig. *C* being tried with the bents about 7½ feet apart lengthwise. In 1883, however, 6400 lineal feet of dikes went out in floods, and the remainder was all damaged, the type of Fig. *B* failing from weakness, and others from scour on the downstream side (owing to the absence of foot mats), or from erosion at the ends; or from drift. The failures, however, were chiefly due to scour, holes being found on the dike lines as deep as 80 feet. Where not destroyed, the dikes were successful in causing deposits from 16 to 30 feet high in two months. Work was continued for several years, the type of dike being modified, but much loss occurred annually, due to scour, drift, and washouts or flanking at the bank. One modification tried was to make the dikes low in the middle and rising at the ends, in the belief that drift would thus be drawn to and pass over the center. This, however, proved an illusion. The work was often carried on also under great difficulties, as at Osceola cross-dike in 1884, where at one time the water was 50 feet deep, with a current of 10 miles an hour, and piles up to 85 feet in length had to be used. At some of the dikes the drift in floods reached a depth of more than 18 feet, the pieces being carried under by the current until the mass was solid to the bottom of the river. Where the structure was strong enough to hold, this acted as a foot mat and prevented too great scour.

The last type used is shown in Fig. *D*, Pl. 18, the number of piles having been increased from the single row of 2 to 5 or 6 rows of clusters, all thoroughly braced. The piles were driven through the foot mattress, which extended 10 feet above the upstream row and 20 feet below the downstream row. The shore ends were revetted for an equal distance above and below, from 250 to 500 feet, or as far as considered advisable. It should be noted that the preceding types had no downstream mats, and that there the damage from scour usually began.

This Plum Point system was finished in 1884, 18,000 lineal feet of dikes having been constructed. Work was then suspended for some years for lack of funds, but what had been done, in spite of all loss, tended to concentrate the flow into the main channels,

and had in general improved the section. It is worthy of note that most of the dikes were on the concave banks and hence they received the full attack of the current and of drift, tending to undermining and flanking. At Plum Point, however, where the dikes were on the convex bank, these troubles did not occur. By degrees it became evident that spur-dikes of the permeable type would be preferable to the longitudinal type, as they produced less tendency to scour and caused deposits to form above and below, and where used to close a side channel, the river would gradually build out upstream and make a new and improved bank line. This it did along the Osceola-Bullerton front.

During the interval from 1884 to 1888 practically no work was done owing to failure of appropriations, and much damage resulted from floods. In 1889 two additional cross-dikes were begun in Gold Dust chute in order to close it further, but scour and drift again did much damage, in spite of the better type used. These were the last dikes built in this portion of the river, and few repairs were made after 1892. The Plum Point system, being on the convex side, built up the bank and was a great success, as was also the Osceola-Bullerton system, which had practically closed the side channel and reduced it from nearly a mile in width at summer stages to an opening of a few hundred feet.

As the Gold Dust chute was still open, a brush and stone dam about 2900 feet long was built across the narrowest part in 1893. (Pl. 18.) This required nearly 20,000 cubic yards of stone. The top was placed 16 feet above low water and the dam was from 3 to 13 feet high, built on sand and silt, foot mats being used below where the water was swiftest. The total cost was $93,000. During the first flood the dam caused a head of 3 feet, and began to settle where on the silt foundation. During the next season it was found to have settled about 2 feet almost throughout, and later it broke in two, a hole scouring on the downstream side with a maximum depth of 65 feet and 3 acres in area. The dam was repaired, but by 1899 it had broken at another place. It was again repaired and built up to the 16-ft. height, but the next year it undermined where on silt, and this part of the dam was seen to settle down and disappear. The brush and poles were rotten and this portion all went to pieces, and in 1900 the chute carried probably as much water at medium and higher stages as before improvement. It became apparent from this experience that better results would have been obtained by a progressive closing, by building up the mattresses gradually, thus largely avoiding the hurtful local currents and the head produced by the dam.

A summary of the experience on this reach, which led to the adoption of dredging as a means of channel improvement, is given below.*

"When the work began nothing was known as to how this river would lend itself to regulation; its power was greatly underestimated and a very large part of the first work was, in consequence, weak and inefficient and was soon destroyed. Just as the

* Copied from an unpublished Report by courtesy of Major E. Eveleth Winslow, Corps of Engineers, U. S. Army, 1900.

river was beginning to be understood, there came a delay of four years (in appropations) during which considerable changes took place in its regimen, and much of the good of the previous work was lost. Since the resumption of work operations have proceeded at a slower rate, and with the design of ultimately completing the regulation of the reach, utilizing as much of the old work as possible. In the beginning operations were carried on at somewhat widely separated places, and the intervals between being regulated have allowed changes to take place that have almost done away with the usefulness of some of the work already done. At the present time several miles of bank are still unrevetted and caving, and some contraction work remains to be done. The object of all this work was of course an increase in the navigability of the river, and, as tending to this, the fixing of the channel in a permanent location.

"The results have been as follows: Before the work began low-water channel depths of 5 feet or less were not uncommon in the reach, but they have not been found since. As late as 1890 the depth at one of the crossings was but slightly over 6 feet, but since that time there has at all times been available a channel of several feet greater depth. Before the work began, Plum Point Reach was known and dreaded as the worst part of the river, and it has been stated by residents of the locality that it was no uncommon sight to see as many as six steamboats aground at one time in different parts of the reach, none of them drawing over 6 feet. At that time the river men would reckon on not more than 8 or 9 feet on this reach when the Cairo gauge registered 20 feet. Now, even at the lowest stage, the river boats can find that depth without difficulty, and for many years navigation has not been interrupted by lack of sufficient channel depths in the reach, which no longer bears the reputation of a bad piece of water.

"In fixing the channel location the success has not been as marked, for in spite of the bank revetment the channel has continued to wander from place to place. In Ashport Bend the channel has always followed the caving bank, but beginning with Gold Dust Crossing, and from that point downstream, the variation in channel location and in the shape of the bars has been very marked. The change in location of Gold Dust Crossing seems to have been largely due to the changing saphe of the bank in Ashport Bend, and less directly to the growth of the head of Elmot Bar due to the sudden widening of the river. Since Ashport Bend has been held in a fixed location by revetment, Gold Dust Crossing seems to have a tendency to become fixed, this tendency, however, being retarded by the hardness and consequent slowness in the erosion of the bar at the head of Elmot. The changes in Fletcher's Bend and below have been due to the slight curvature of that bend, to change in the point of impingement of Gold Dust Crossing, and to caving of the bank at the point of impingement and the consequent changes in the shape of the bank of this bend.

"From the experience gained in this reach it may be stated that the following facts appear to be true: That the banks of the river can be successfully revetted; that side chutes can be successfully closed and that the river can be otherwise contracted

where necessary; that these works both of revetment and contraction will be expensive; that an efficient and permanent regulation is not possible except by bank revetment, but that contraction will also be necessary in places; that the sequence of changes going on as a result of caving banks will continue for some years after the bank is held, and that in general the full results of work of either class will not be shown for several seasons; that permanency of location will be more easily obtained the greater the curvature of the bends and the more regular this curvature; that in systematic regulation downstream, and in general, the complete regulation of the lower Mississippi will be a work of vast magnitude that would at best extend over a long series of years, and that therefore the immediate needs of commerce must be supplied by other means, applicable at short notice, to remove local impediments."

Experience on the lower Mississippi indicates that with rivers of strong currents the dikes should be placed if practicable on the convex shores, and should be permeable spurs, so as to assist the natural forces in building out the bank. The concave shore should be held by revetment where necessary, not by dikes, as the latter would be exposed to undermining and destruction.

An experiment on the Indus with the type of dike used in Plum Point Reach, and which was tried because of the work then being done on the Mississippi, will be found described in the next chapter.

**Cost of Dikes, etc.**—The cost of dikes varies within wide limits according to the locality and materials used. In America the materials are usually obtainable near the site of the works, and are of non-permanent character, so that the first cost is comparatively small. The cost of certain typical dikes was as follows.

*Missouri and Mississippi Rivers. Permeable Dikes.*—The cost of 550 lineal feet of a 3-row permeable pile dike (such as has been previously described) on the Missouri River in 1900, was about $11 per lineal foot. This included all expenses except those of engineering and office work. The piling and bracing cost in place about 50 per cent of the total, and the mattress and pole work in place, about 26 per cent. In other localities estimates for 3-row dikes were taken as $20, and for 5-row dikes, as $35, per lineal foot, all included. The distribution of field expenses was found to be about as follows: For actual construction 67 per cent; for care, repair and moving plant 22 per cent (this includes an item of only 5 per cent for light repairs); administration 9 per cent; and 2 per cent for all other items, including surveys and travel. The cost of several 3-row dikes built in 1909–1910 is given as from $10 to $16 per lineal foot.

The cost of permeable dikes of somewhat similar construction on the Mississippi near St. Louis ranged from $10 to $35 per lineal foot, according to strength.

The cost of the brush hurdles or abattis (Fig. 83, p. 161) used on the lower Mississippi and the Missouri to close side channels is from $4 to $5 per lineal foot.

Between 1878 and 1904 there were constructed on the upper Mississippi 231 miles of dikes, of the type shown in Fig. 91, p. 170, and containing 8,941,000 cubic yards of

brush and stone. The average amount of rock used was 2.97 cubic yards and of brush 4.34 cubic yards, per lineal foot.* On the basis of prices obtained in 1905, for contract work in place, the average cost per lineal foot would sum up to about $6.40, not including any office expenses.

The cost of the 1200 feet of concrete-pile permeable dikes built during 1909–1910 on the Republican River in Kansas (see p. 170) was as follows:

| | |
|---|---|
| Making piles, 7270 lineal feet at about 61 cents | $4,421.00 |
| Driving piles (by contract), 6966 feet penetration at 31½ cents | 2,194.00 |
| Concrete braces in places, 6516 lineal feet | 2,890.00 |
| Screens for 1200 lineal feet of dike | 256.00 |
| Foot mattress, 63,000 square feet (50 feet wide) | 3,529.00 |
| Sundries | 358.00 |
| Total at $11.37 cents per lineal foot | $13,648.00 |

The estimated cost of wooden pile dikes for the same locality was about two-thirds of that for the concrete.

*Arkansas River—Mattresses* (see p. 163.)—Amount of wire used per 100 square feet, 1.6 pounds to 5.3 pounds. Average 2.5 pounds; brush per 100 square feet, 0.42 cord to 0.65 cord. Average 0.52 cord; stone for sinking, 1 cubic yard per cord of brush.

Cost in place of mattresses 60 feet wide, including stone revetments, $3.28 to $4.16 per lineal foot. Average $3.93. Cost in place, mattress and revetment together being from 150 to 175 feet wide, $11.31 per lineal foot, the riprap being carried up to a 10 foot stage. Cost in place of mattress 120 feet wide, with revetment carried to a stage of 15 feet, $14.52 per lineal foot.

*Dikes.*—Cost of permeable pile dikes, $6.86 to $23.36 per lineal foot. Average $15.68.

Cost of pile-and-stone dikes in similar depths of water averaged $26 per lineal foot.

Stone dikes in water 2 to 5 feet deep at low stage, $10.50 per lineal foot; in water 25 feet deep, $76 per lineal foot; in water 40 feet deep, $81.60 per lineal foot.

The above costs include care of plant during periods of idleness. The net costs would be about 30 per cent less.

*Pile and Brush Dikes on other Rivers.*—The cost of the small pile-and-brush dikes described on p. 152, where materials were cut on the banks close by, has ranged from $1.50 per lineal foot up, and that of pile-and-plank dikes from $3 per lineal foot up.

* Annual Report, Chief of Engineers, U. S. Army, 1905.

# CHAPTER V.

## PROTECTION OF BANKS

**Objects.**—The purpose of the protection of banks is the prevention of erosion, and may include one or more of the following objects:

1. The reduction of the quantity of traveling sediment and consequent lessening of deposit in the river-bed.

2. The maintenance of a permanent minor bed of normal depth and width in the bends.

3. The protection of property, wharves, landings, etc.

4. The protection of levees built along the banks.

5. The prevention of cut-offs which may affect a river's regimen and also leave important commercial centers without means of transportation.

Rivers in flood times usually display a constant tendency to cut away the banks, particularly on the concave sides at bends. The material thus eroded is moved along until it finds lodgment, and may later obstruct low-water navigation in the form of a bar or shoal. As the bank is cut away the river shifts its position toward that side, and year by year its low-water channel there, as well as its bank line, is moving, and it is necessary, in order to produce a more stable bed, to permit a minimum of encroachment upon the banks and movement of the channel. In other words, the shores should be held to a fixed line, and thus give a permanent, if restricted, passage for the water, and one which will tend to a less degree to silt up with material cut from the banks.

**Theory.**—The theory upon which the use of revetments, protection spurs or dikes, and bank protection in general is founded, is illustrated by the accompanying cuts. Fig. 93 shows the centrifugal tendency of the current in passing a bend; it is thrown against the bank, and the reaction caused by its being checked causes a boring or eddying action, which erodes and carries away any light material. Fig. 94 shows the profile of a bank which the river is beginning to attack, such as may be seen at the beginning of a bend; Fig. 95 shows the erosion in progress, and Fig. 96, the final stage when the bank has been cut back till it stands almost vertical. The entire river-bed shifts towards the concave bank, since as fast as it is eroded, the opposite or concave bank is built out, the river thus preserving an approximately constant area, as explained in Chapter I (p. 23.) It may be noticed that the material at the foot of the bank below the water-line is shown as being on a slope; this is due to the fact that it usually falls in more quickly than it can be removed, and it is only under comparatively stable banks

that a steep face above water is accompanied by a similar one below. The falling-in of the upper portion of the bank is due primarily to its attempting to stand at an angle almost vertical, or at least far beyond its natural slope of repose, and hence it breaks off piece by piece as shown by the dotted line in Fig. 96. This breaking is most severe after wet weather, or when a flood has saturated the earth and has receded quickly, leaving a weight of water in the bank, which was scarcely able to support its particles even under ordinary conditions. In protecting any bank, it is therefore a necessity to reduce it to a moderate slope which will not slide when saturated. The cause just mentioned—saturation—frequently occasions breakages and sliding of banks of natural slope and where no erosion is in progress. After the sudden recession of a flood there may at times be seen considerable portions of banks cracking and slipping down, sinking at some places only for a foot or two, at other places till the slide disappears below the

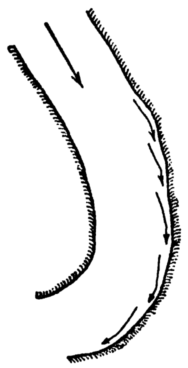


FIG. 93.

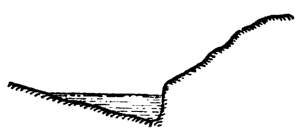


FIG. 94.

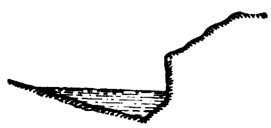


FIG. 95.

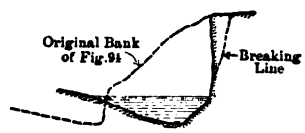



FIG. 96.

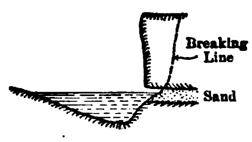



FIG. 97.

water, and sometimes the trees upon it can be seen standing in the current a hundred feet or more from the bank.

The breaking is accelerated in many cases by the presence of seams of sand near the foot of the bank which seep out and leave the earth above without support, as shown in Fig. 97. Cases of this nature are to be seen on all alluvial rivers, and are a fruitful source of caving, especially where a body of water, as a lake or an old river-bed, lies behind and near the bank and filters through it into the river. Where the upper stratum is of compact material, the bank may settle vertically instead of breaking off, as occurred at Old Town Bend on the lower Mississippi, where a section a mile or more in length and 200 to 300 feet wide settled as much as 15 feet in one season, but so evenly that the general surface was only slightly cracked, and the trees were hardly disturbed. Such a slide may remain for many years before it is entirely washed away and the stratum of sand exposed again, but as soon as this takes place a new settlement will begin.

The presence of large trees along a bank will also assist its breaking, partly by their weight, but chiefly because of the straining and leverage they cause in the soil during high winds.

Where a hard stratum, such as clay or rock, exists instead of earth, the erosion is slight, and usually occurs during high water only, the bank then taking a profile as shown approximately in Fig. 98. Of this condition examples are usually to be met with on all alluvial rivers. Nature thus shows that if the toe of a caving bank can be held, or if the erosive action can be kept at a distance from it, the danger of caving will be reduced to a minimum. This holding can be done, as described further on, by covering the surface of the bank up to high-water mark with non-erodible materials, or by breaking the attack of the water by means of spur revetments or spur dikes, or by guarding the toe with a longitudinal dike of stone or piles. (See Figs. 99 to 101, p. 183, for types used in Europe.) In every case the bank must be graded to a flat slope, and precautions must be taken to guard against the erosive action always present along the toe, and the work should extend above and below the stretch where erosion is going on. If this cannot be done, the up- and downstream ends must be thoroughly protected by riprap or by other means against undermining and scour, as must also the longitudinal portions under water—the outsides of mattresses or the piles and the ends of spurs—along or around which there will be at all times a constant stream of eddies. The protection must also be carried up to or above high-water line, or the floods will undermine it along the upper edge. The depth to which the scour is likely to occur may be found roughly by taking soundings in neighboring portions of the river and finding the deepest trough at which conditions as to curvature, soil, etc., are somewhat similar. If the proposed construction will not break the continuity of flow nor cause unusual swirls or reactions, it is not probable that it will cause scour to a depth greatly different from the maximum thus found.


FIG. 98.

**Surface Covering.**—Where a friable bank is exposed to wave action or to strong eddying currents, it is usually advisable to employ a close surface covering in addition to the general protection. When the eddies or ripples can work through the interstices of a mattress or of riprap, they will remove the soil bit by bit until cavities exist which will eventually cause partial or entire failure, and the only security lies in protecting it from the force of the current so that if the eddies or ripples reach the natural foundation, the disturbance will be too weak to cause erosion. A surface of fine sand will lie undisturbed under a smooth current of considerable velocity, but as soon as it is exposed to water charged with swirls and eddies such as the rough exterior of a mattress or riprap revetment is always liable to produce, the particles are picked up and carried away. Hence arises the necessity of keeping at a safe distance the disturbances of the current. Where the work lies under water, reliance must be placed upon the density of the mattresses or upon the quantity of riprap to avoid these dangers. On the lower Mississippi, where the cost of riprap is high, the mattresses, covered only with enough riprap to sink them, usually constitute the entire protection, and the fascine type has been found

much preferable to the plain woven or crossed type, as it forms a denser mat and is less liable to tear apart if the water should form occasional cavities beneath. Where the work is above low water, a covering of gravel or spalls, which should not be less than 4 inches deep, can be spread under a surface protection of riprap. When the ripples or waves splash through the openings in the latter, they will then expend their force on the covering beneath and thus fail to reach the soil. (See Fig. 102 for a type used in Europe.) If, however, the gravel or spall covering is omitted, the soil will be washed out and cavities be formed which may ultimately require the regrading and riprapping of the whole bank.* In Holland clay is often used for this purpose, a layer a foot or more in thickness being placed on the graded banks, which are usually composed of fine sand. The clay is then covered with straw and a paving of concrete or stone with close joints put on as the final protection. In many examples the concrete has been used, laid in separate slabs about 6 inches thick, $5\frac{3}{4}$ feet wide (measured up and down the bank) and $7\frac{1}{4}$ feet long, with a space about a foot or more in width left on all sides. These spaces were filled later with reinforced concrete, so that the original slabs lay as it were in a framework, their edges being tongued or shaped so that they could not be lifted out of the frame. The cost of this work is stated to have been about $1.18 per square yard. (See also p. 186.) On the Volga a thick covering of straw is sometimes used, laid directly on the soil and covered with riprap, and in bad places two such layers of straw with riprap are employed.

Attention to this feature of bank protection is of the first importance, since the worst effects of the eddies or waves usually occur in floods, when repairs are difficult if not impossible to make, and very serious damage may be done before the flood recedes and permits steps to be taken to remedy the danger.

It is rarely necessary to carry the protection to the full height of the banks unless they are considerably below the level of the highest floods, and recourse is had in many cases to sodding the upper portions. In America this has been done only to a limited extent, but the practice is quite common abroad. This kind of revetment is made by means of pieces of sod cut into squares or rectangular figures and placed in courses normal to the slope where the latter is steep, and parallel to it where it has a gradual inclination. In certain localities, as on the Mississippi, Bermuda grass is used, the sprigs being placed from 6 inches to a foot apart. As this grass possesses a phenomenal vitality, the roots spread rapidly and form a dense sod in the course of a year or two, completely covering the bank.

**Types Employed.** (See Figs. 99 to 103.)—It is important in laying out protecting works that they should be planned so that they will not modify too greatly the regimen of the river and thus bring currents upon points heretofore unattacked. While riparian owners are compelled to endure losses from floods, they have good grounds for complaint when the construction of works brings injury upon them. In fact the Government

* See also remarks on protection of banks in Part II, Chapter IV, "Fixed Dams."

FIG. 101.—Cribwork of Fascines.

FIG. 102.—Stone Paving on Gravel Bed.

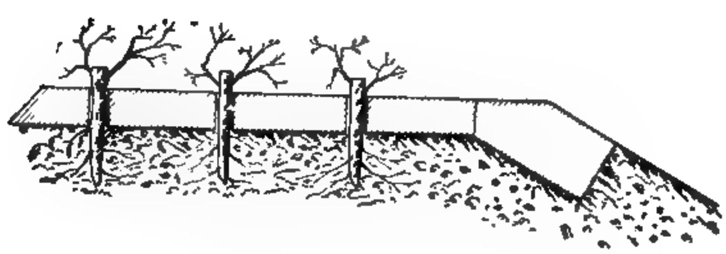


FIG. 103.—Stone Paving, with Willows.

SECTIONS OF BANK PROTECTION.

is often held responsible in the public mind for damages resulting entirely from causes over which it has no control, if such injuries have occurred in proximity to public works. It is thus of first importance to have a thorough knowledge of the reach to be protected, as time and money both may be lost through a lack of acquaintance with the currents, etc.

The revetments constituting the protecting works of river banks comprise in general two distinct forms. In the one the bank is covered completely for the entire length of the space to be protected; in the other, the space is divided into sections by separate revetment-dikes placed at considerable distances apart, the bank between being left uncovered. The revetments themselves may also be considered as of two distinct parts, one below and the other above low water; the first is inaccessible and invisible, and must protect the second, which is exposed to all the changes of weather and water, and is open to inspection during the dry season. As the portion under water must act as a foundation for that higher up and cannot be readily inspected and repaired, it should be built in a substantial manner. A stable foundation is a requisite for good bank protection, since if it is composed of soft or perishable material the destruction of the upper part will assuredly result. Where practicable it is usually composed of stone thrown into the water and allowed to take a natural slope. In many cases the pieces will settle into the river-bed, but this settlement will eventually cease as additional material is put in. To avoid this subsidence a trench is sometimes dredged into which the stones are dropped, forming a toe. This, if carried to a sufficient depth, insures permanent stability for the revetment. Somewhat similar methods have been employed with much success by engineers in India, as described elsewhere in this chapter.

The kind of protection to be employed at a particular locality will depend on the regimen of the river and the cost of materials. A stream of moderate current can usually be successfully treated with works of light construction, while one of deep, swift current will require much stronger and more extensive works. The thin paving of stone protected above water by willows, or the graveled slopes protected by sod, or the clay-weighted fascines which are to be met with in Europe would prove sufficient for some rivers, while for others large fascine mattresses with riprapped slopes would be necessary. In many cases, however, brush may be inaccessible, and climatic conditions, as in India, may render its use of doubtful advisability. Where stone is abundant, it will usually give the most satisfactory results, since the material is permanent, and if used in sufficient quantity it will adapt itself to all ordinary changes which the river may produce on the surface of the bank. The two principal matters to bear in mind in using any type of protection are, to guard against undermining along the top and bottom and at the extremities, as well as between the interstices of the material, and to present as even a surface to the current as possible, so that the water will not form any objectionable eddies or reactions which might tend ultimately to injure the protection.

*Piles.*—The use of round and sheet piles in connection with broken stone is frequently met with, and their employment usually reduces the quantity of stone required, and if they are driven well below the limit of erosion assists in protecting the work above water against the danger of future settlement. A base is also obtained in this way which assists in distributing the loads over soft soils. Whatever woodwork is used in the foundation should be constantly submerged as far as practicable; if timber must be used in exposed positions it should be where it can be inspected and repaired during low-water seasons.

Where conditions are such as to prohibit the use of spurs or of mattresses, a comparatively economical protection can sometimes be obtained in this way by driving a row of sheet-piling or of close round piling along the toe of the bank. (Fig. 99.) The piles must be long enough to prevent any undermining, or else plenty of riprap should be used along the outside, and if round piles are used, it is usually necessary to place brush behind them to prevent the soil from the bank being washed through. Where there is liable to be much pressure from the bank, it may be desirable to use an inner and an outer row, tied across at the top, the space between being filled with heavy material, so that the structure will act as a retaining wall. With a light pressure only, tie-piles in the bank at distances of 8 to 10 feet will usually provide enough support. Concrete-steel piles have also been employed in these situations, but they are expensive.

*Masonry and Riprap.*—A revetment of dry masonry, or of masonry laid in mortar, is sometimes used as a foundation, and for the protection above low water as well, but it is of course more expensive than loose stone or timberwork, and in the majority of cases is more difficult to build and repair. Brick and concrete are also employed. With all these types it is usually necessary to employ a protection-apron of broken stone to prevent erosion along the outside. Masonry protection works, however, whether of stone, concrete, or brick, are not generally used except at cities or great industrial works, on account of their cost. They give satisfactory results, both in permanence and in appearance, and present no projections to damage craft, but where on soft foundations a settlement often occurs, breaking their continuity and not infrequently resulting in serious consequences. This is particularly noticeable in masonry where mortar is used. The solidification of the mass by the mortar may permit a large area of earth underneath to give way before failure, and thus there may occur without warning a break of considerable dimensions, and one which may be left unprotected, or be even unknown during lengthened periods of high water. Where there is no mortar in the joints the paving will generally follow the sinking of the ground and keep it covered, and, if not submerged, will at once reveal what is taking place.

When riprap is employed the pieces are sometimes placed flat, in thin layers, and held in place by vegetation, in order to reduce the quantity of stone required. This method has been used in France to a considerable extent. Usually flat stones are used for this work, but along the Meuse " dog's-heads " (cobble-stones) are frequently employed.

In placing the pieces they are usually laid side by side on the graded bank, the joints being filled with thin slips of willows. If the work is done at a favorable time the willows at once take root and protect the stones by both branches and roots, thus forming a perfect connection between the bank and its protection. These works are said to resist ice, the great enemy of bank protection in many localities, better than the masonry protection above described. In order to keep the willows in bush form and thus secure the best results it is necessary to cut them from time to time. It is claimed for this form of revetment that it is at once economical and efficacious (Fig. 103).

On some of the smaller Indian rivers work exposed to very violent currents has been protected with cobblestones and boulders encased in a wire netting. These " sausages," as they have been termed, are usually made a foot or 18 inches in diameter, and up to 40 feet in length. They have proved very effective when used in sufficient quantity. (See also p. 154.)

*Concrete and Concrete-Steel Revetments.*—Revetments of these types have been applied on the Dortmund-Ems Canal since 1894, on the North Sea-Baltic canal, on the Mississippi, in Holland, (see p. 182) and elsewhere, with considerable success. The concrete is usually placed on a layer of broken stone about 4 inches thick, or on a bed of clay, and in some cases piles of wood or metal, from 2 to 5 feet long and 2 to 3 feet apart, are driven into the bank, and the concrete is placed around their tops. With piles no broken stone is used. The concrete in the European examples is usually composed of one part of cement and 3 to 12 parts of sand and stone, according to the exposure, and is from 3 to 7 inches in thickness, with reinforcing rods $\frac{1}{8}$ to $\frac{1}{4}$ of an inch in diameter, and 6 to 15 inches apart in each direction. Experience has shown that it is best to introduce joints for temperature every 20 or 30 feet. The cost of these revetments is given as from $5\frac{1}{2}$ to to 9 cents per square foot.*

The principal objection to this type of protection, as with that of solid masonry, is that it is liable to be undermined by rain, springs, or similar causes. Cavities are thus formed underneath, and when the concrete becomes too weak to support its weight it breaks and falls, leaving the bank partially exposed. This has been the experience on the Dortmund-Ems Canal, the Mississippi, and elsewhere. For river work the riprap type of protection appears preferable, as it is less liable to such accidents and is more easily repaired.

About 1890 a type of protection was introduced in Italy composed of blocks of terra-cotta or concrete, strung on wires, as shown by Figs. 104 and 105. It has since been used on the Po, the Nile, the Dee, and other rivers, and is reported to have given satisfaction. The blocks are about 10 inches wide and 4 inches thick, and weigh from 25 to 30 pounds apiece.

FIG. 104.—Isometric View of Block Protection.

* "Le Béton Armé," Paul Christophe. See also p. 182.

The wires are galvanized, or covered with lead, or made of non-rusting metal, and are attached at intervals of about 6 feet to an anchor pile buried in the bank. This style of covering is laid by means of a floating scaffold and a special winch in elements from 15 to 30 feet in width, each element being continuous with the one adjoining, and the up- and downstream extremities are protected with fascines or riprap.

*Concrete Ballast.*—About the year 1900 experiments were made on the lower Mississippi River with the use of ballast or artificial riprap made of concrete, with the view of using it instead of the natural stone employed in sinking mattresses. As this stone had in most cases to be brought to the river by rail and then to be towed long distances, its cost was considerable. The experiments having been successful, the ballast has since been used extensively where cheaper than stone. The gravel and sand are obtained from the nearest supply and are mixed by machinery, Portland cement being used in the proportion of 1 to 14 or 16. The material is cast in blocks, which, when thoroughly

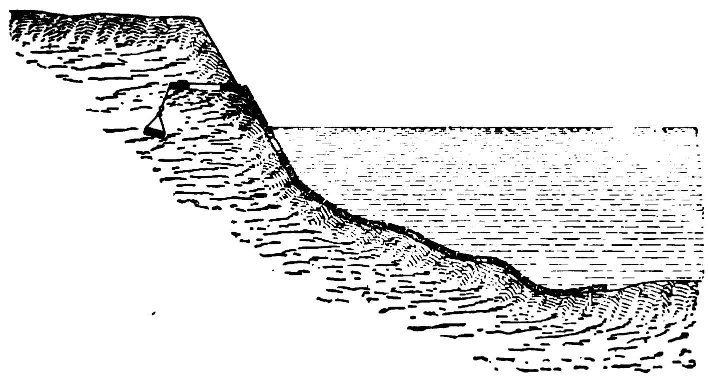


FIG. 105.—Section of Block Protection in Place.

hardened, are broken up by sledges into pieces convenient for handling, and are then placed on the mattresses in the usual way. It is estimated that this breaking causes a loss of about 10 per cent. The cost of 10,683 tons of this ballast in 1903 was from $1.17 to $1.90 per ton, the average being $1.55 per ton. Of this amount the cement cost about 84 cents per ton.*

*Fascines and Mattresses.*—Excellent results are frequently obtained by using various forms of brush, poles, etc., in protecting banks. These materials are cheap and usually grow in profusion near the points requiring works of defense. They are employed in a great variety of forms, but principally in the shape of fascines and pole mattresses. The former are round bundles of flexible branches held together by wires or other ties. They are placed close together, either singly or in layers over the bank to be protected, to which they are held by stakes driven into the soil. They may be made into mattresses by means of poles, or by merely connecting them with wire cables, and when so arranged

* Annual Report, Chief of Engineers, U. S. A., 1903.

they form an excellent covering of great strength, and in streams of moderate current they frequently become filled with deposit.* (See Pl. 21 and pp. 205 and 206.)

The fascines used in Holland are 8 to 13 feet in length, and from 1 foot 4 inches to 1 foot 8 inches in diameter. On the upper Rhine they are from 13 to 16 feet in length, and from 1 foot 2 inches to 1 foot 10 inches in diameter.

Pole or ordinary mattresses are made in numerous forms and by several methods. They consist essentially of poles, brush, branches, etc., laid side by side and woven together with wire, or fastened with timber and sunk into position and held there by stone. (See Pl. 20 and Fig. 111, p. 195.) In America galvanized iron and silicon bronze wire are generally used for binding, as ordinary wire soon rusts out and permits the mattresses to go to pieces. Wooden pins have also been largely employed for fastening binding pieces, and are said to be a satisfactory method under suitable conditions.

On rivers in the Fen districts in England connected fascines each about 6 feet long and a foot in diameter, weighted with a layer of clay about a foot thick, have been successfully used to protect the banks. This method is stated to have an advantage in permitting boats to run into the bank without injury either to themselves or to the protection, an accident which happens not infrequently, as the sailors of the steam trawlers on leaving port are sometimes unable to steer properly.† Mattress weaving as practiced on the Mississippi and Missouri rivers will be found described further on in this chapter, at the end of which details of cost are also given.

*Brush and Cables.*—A temporary protection of a type which was employed on the Rio Grande at the U. S. Military Reservation at Brownsville, Texas, where stone could not be obtained except at a prohibitive expense, consists in wiring the butts of shrubs to galvanized cables running parallel with the river, so as to make a dense continuous fringe, the ends of the cables being fastened to piles or timbers, with the brush pointing down the slope of the bank. Cross cables running up the bank are wired to the longitudinal cables at distances apart from 10 to 30 feet, with their upper ends secured to posts. The first cable is placed at or below low water, so that its brush will be all submerged and lie at the base of the bank, and as the protection of the toe depends on it, it should be well secured and have plenty of material tied to it. The next cable is placed further up the slope, so its fringe will cover the intervening space, and still other cables further up as may be needed. The whole forms a thick brush mat, most of the lower portion of which will rapidly fill with sediment. In one instance the undercutting at the toe caused the mass of brush and sediment to break the cross cables, and the whole slid down the bank several feet, but effectively stopped all further erosion.

The cost of 15,000 square feet of this type of protection at the locality mentioned was about six cents per square foot in place, using cheap Mexican labor.

**Spur-dikes for Bank Protection.**—The generally accepted theory of the action of spur revetments and dikes (which can often be verified by an examination of localities where

*For Lumber Mattresses, see p. 192.

† Proceedings Int. Cong. Navigation, Glasgow, 1901.

such works have been placed too far apart), is illustrated by the accompanying cuts. Fig. 106 shows the effect of isolated spurs when placed on a bank undergoing erosion. Each dike holds the current temporarily off the bank and permits it to take a straight course, approximately at right angles to the radius of curvature at the spur, until it meets the bank again, when erosion again begins. This action may be verified by watching débris floating past a dike located as described. Fig. 107 shows a series of spurs at varying distances apart, and illustrates the fact that for proper protection they must be spaced according to the curvature, and must be sufficiently close to prevent the water attacking the bank before the succeeding spur is reached. Unless this is done erosion will continue in the spaces between the spurs, and the latter may be eventually undermined. The outer ends of the spurs may be stepped or sloped down so as to follow approximately the contour below water and thus protect all parts, or, if built with vertical ends, the spurs must be run far enough into the river to keep the current away from the toe of

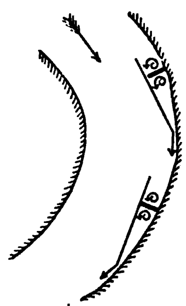


Fig. 106.

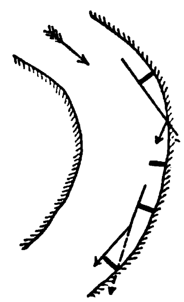


Fig. 107.

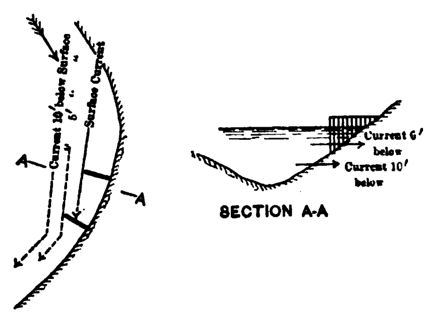



Fig. 108.

the bank. If, for example, the spur is made of a row of piles with tops all on one level, and of such a length that the outer ones just suffice to prevent the water at the surface from attacking the adjacent downstream bank, the slope of the latter below water will gradually become again exposed to the current with the increase of depth, as illustrated by Fig. 108, and will be eroded as before unless further protection is used.

For small rivers, spur-dikes of brush and piles, a description of which was given on p. 152, will often be found a cheap and effective protection. They should have stout timbers bolted along the tops of the piles, and be well braced if they have to stand blows from drift. Two rows of piles, filled between with brush, are preferable in exposed situations to a single row with a brush screen, as the latter would soon be destroyed by drift. This permeable type has one great advantage over a solid dike, in that it permits the water to pass through it and thus produces less disturbance of the current, and hence where a solid dike would create eddies which would result in more or less scour around it, a permeable dike will slacken the current and cause it to produce deposits. (See also Chapter IV.) The portion of the bank adjacent to and above each spur should

be graded to a flat slope and be covered with a revetment, or the water will tend to cut around it.

The spur method of protection, where used on a concave bank and in a strong current, is not usually looked on with favor, as the projection of the dikes directly into the current causes a reaction which tends to undermining and erosion (see p. 144), and the additional material necessary to prevent damage from this cause has in some cases rendered the total cost considerably more than would have been that of a protection laid directly upon the bank. The efficiency of this type when placed in caving bends is, as above described, chiefly dependent upon the radius of the bends, the distance between the spurs and the material of which the bank is composed. They are stated by the Mississippi engineers to be of little value in abrupt bends with sandy banks, and that where the banks are of clay and "buckshot," the distance apart should not exceed 500 feet, and even then it may be necessary to protect the intervals between them. Where built in moderate currents, however, they will often cause a filling out of the bank line, and prove very beneficial. (See Pls. 3, 4 and 5.) For this reason they can often be effectively employed in the tidal portions of rivers, and in protecting shores from erosion by waves. Where used for the latter purpose, they appear to give the best results when made of small height and placed near together. Thus at Fort Caswell, on the Atlantic Coast, after an attempt to protect the property by a seawall had failed due partly to undermining by the waves and partly to the sand under the foundation being blown away by high winds, a series of spurs or groins was laid on the foreshore. They were composed of bags filled with sand, and the total height of each groin was about a foot. These offered so little obstruction that instead of causing scour, they brought about an immediate filling-in of the shore line, and as soon as they were covered by the sand they were raised and extended, resulting in a steady building up of the shore.*

An unusual type of spur protection has been occasionally used on the Missouri River, consisting of series of superimposed pens made of brush, something like a series of empty boxes placed end to end and one on another, the boxes being 5 feet square inside, or more. The first layer is sunk to the bottom, being held to the bank by wire cables, and the pens soon begin to fill with silt, thus anchoring the structure securely. Other layers are placed on this foundation in a similar manner up to low-water level, and the whole soon becomes weighted in place with silt. Above the water level the bank is protected with riprap. The pens are usually stepped back as they are built up, so as to offer less projection to the current.†

*Spurs at New Orleans and Memphis.*—In 1884 a continuous revetment in New Orleans harbor was broken up by the river after sinking, and this led to the introduction of submerged spurs normal to the bank and placed at intervals of from 500 to 1600 feet, usually at salient points. These structures consist of a woven mattress foundation of the width

* Annual Report, Chief of Engineers, U. S. A., 1903.
† For a general description see "Engineering News," October 22, 1908.

deemed advisable, and extending out into the stream usually beyond the deepest water and sometimes protected on the edges with a narrow cribwork of willow poles filled with rock. In one case additional cribs were sunk one on top of another, at a distance of about 70 feet below the upstream edge, affording a base of about 60 feet and a top width

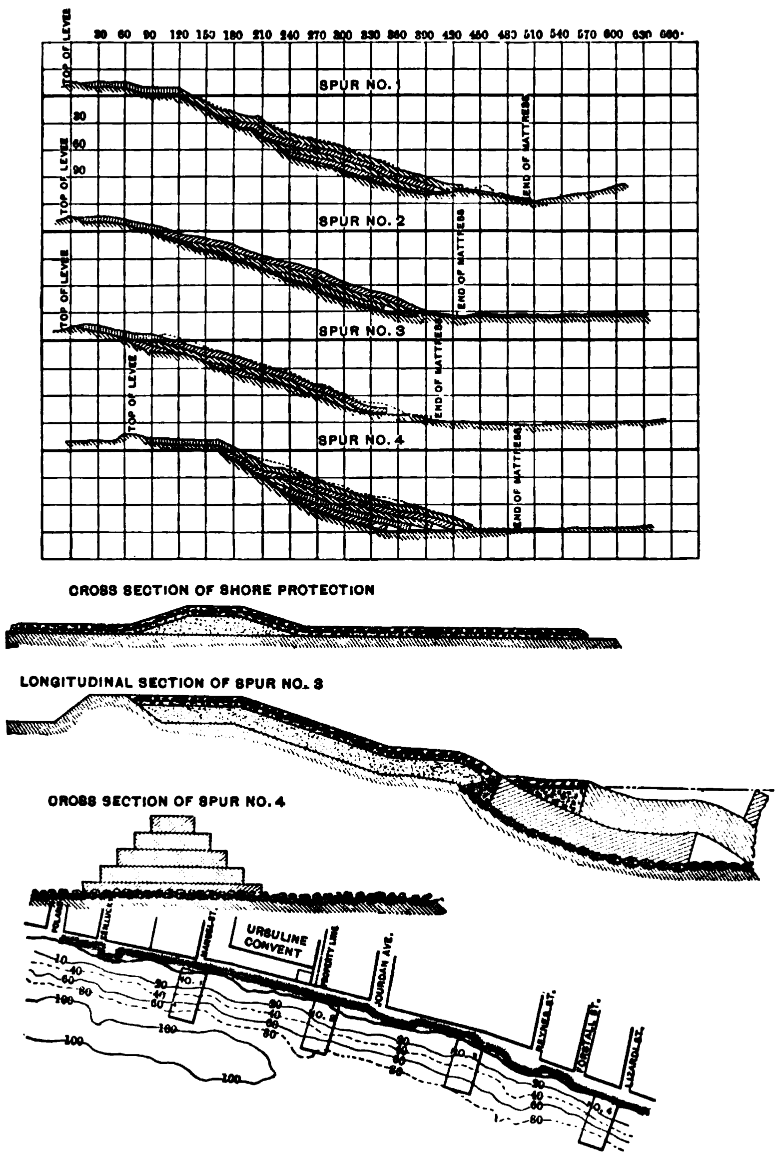



FIG. 109.—New Orleans Spur Revetments. (For Bank Protection.)

of about 22 feet. (Fig. 109.) This cribwork was about 300 feet long. Similar work was begun at Memphis in 1886, when severe erosion was taking place along the upper end of the city. The foundation mattresses were made 200 feet wide and placed from 450 to 525 feet apart, extending from high-water line into the river for a distance of 350 feet from low water. On these, cribs of poles were sunk, their centers being placed about

70 feet below the upstream edges of their respective mattresses. The cribs were made about 8 feet wide and were placed in rows side by side. The distances between the spurs, however, were found to be too great, and in the year following intermediate ones were put in. This protection has been successful in its object, although, owing to the strong currents, many repairs were at first required, and it gradually became evident that a cheaper method was to be found in revetment placed directly on the banks. Such spurs are now rarely used along this and similar portions of the Mississippi except to break up eddies, and in this they have proved very successful when well protected by foot mats. The work at Memphis is shown on Pl. 19. The irregular bank line produced by the eddies around the dikes is worthy of notice.

Some open spur-dikes of piles were used at New Orleans in 1897, the piles in the rows being placed about 10 feet apart, measured at right angles to the bank, and extending out into the river to a depth of 15 to 20 feet below low water. These dikes reduced the current, causing a deposit of sediment and a filling out of the bank line, but in the upper portions of the river, where the currents are of greater violence, they would have been liable to destruction by drift and floods. This type of open or permeable protection appears in fact to be successful only in moderate currents, unless extraordinary means are taken to prevent undermining. It was tried as a protection in one length of 4600 feet on the Indus in 1901, after an examination of similar work in America, with the result that the piles and connecting work were practically demolished by undermining in the first flood.*

**Lumber Mattresses.**—In 1897 a novel type of mattress was introduced on the middle portion of the Mississippi River, composed of planking. The first ones were used near St. Genevieve, Missouri, and Beach Ridge, Illinois, and gave such satisfactory results that they have largely replaced brush mattresses on certain portions of the river. They were found to possess marked advantages in construction and handling, being cheaper in cost of material and requiring only half the number of barges, etc., while twice as much protection could be laid with them as with brush mattresses in the same time. Their durability and resistance have so far proved very satisfactory. However, where the currents are strong the many interstices between the pieces are liable to be a cause of undermining, just as was found to be the case with the pole mattresses used on the lower Mississippi.

The general method of construction and launching is the same as described for brush mattresses, and the lumber used is of the cheapest class, the only requirements being those of soundness and freedom from large knots or other defects which would cause the planks to break when bent. It is used in pieces from 4 to 6 inches wide and 1 inch thick, and in lengths not less than 12 feet. Construction is begun by laying across the foot of the ways on the barge two or more thicknesses of plank, well fastened together, to form the under half of the "head block," or outside stringer. At right angles to this and

* "River Training and Control in India." See also p. 143, and elsewhere in Chapter IV.

parallel to the current are placed selected " weaver planks," about 3 feet 9 inches between centers, their ends being fastened to the head block, which is then completed by fastening to it, by nails and wire, a piece similar to its lower half. The mattress planks are placed to pass alternately over and under the weavers, leaving a space of 4 to 5 inches between each plank, and the whole is tied together by additional pieces, as shown on Pl. 20. Care must be taken in launching to see that no pieces become broken, and if this occurs they must be repaired, or trouble may result later.*

With a total of seven miles of mattresses, an average of 210 linear feet was placed per day. The mattresses were 125 feet wide, and one section was built in an unbroken length of more than 10,000 feet. A total of 86 men were employed, exclusive of cooks, etc. Details of cost will be found at the end of this chapter.

**Use of Anchors.**—The practice of using piles to which to secure mattresses while being sunk in a current has in many cases been replaced by the use of cast-iron mushroom anchors, sunk into the river-bed by means of a water jet to depths ranging from 10 to 20 feet, according to the material and the strength of the current. Those used in this manner are 16 inches in diameter, and are placed from 25 to 40 feet apart, being provided with staples to which the mooring cables are secured. When finished with, they are loosened by a water-jet and pulled up. A single 20-inch anchor on one occasion held a large dredge and several other boats in a strong current.

These anchors are much cheaper than the standard anchor piles, and on the advent of a sudden rise do not form a danger to boats.†

**Bank Protection on the Mississippi.**—Mattress revetment is the chief method of bank protection employed along the Mississippi River. The brush grows in abundance, and in spite of continued denudation for these works the supply has not diminished, as the cotton-wood and willows spring up rapidly after being cut down, and form the cheapest material for use. Out of the abundance and cheapness of this material has grown the practice of its use, in connection with stone, also fairly plentiful, as a revetment for banks in the United States.

The general features of the Mississippi have been described in preceding chapters (p. 86 and elsewhere). As therein stated, the river is divided naturally into three parts, the first extending from the headquarters to the mouth of the Missouri, and possessing a comparatively stable regimen; the second, in which erosion is more marked, from the mouth of the Missouri to the mouth of the Ohio; and the third, from the mouth of the Ohio to the sea. The last section is one of strong currents and unstable banks, except in its lower portion, south of Baton Rouge, where the regimen becomes more settled in accordance with conditions usually found as the mouth of a river is approached.

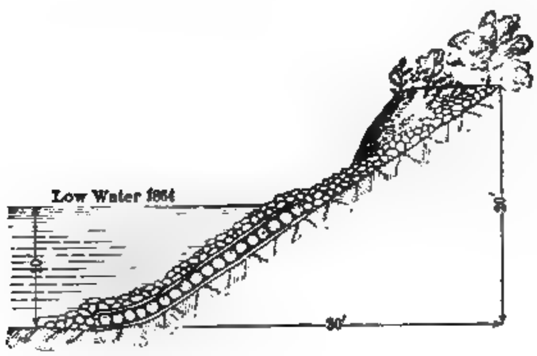
The chief types of bank protections used have been the pole and the fascine mattress, and in recent years, the lumber mattress, described on p. 192. (See also Pl. 20.)

* A detailed description of the methods used will be found in the Annual Report of the Chief of Engineers, U. S. Army, 1901, pp. 2215–2218.

† Annual Report, Chief of Engineers. U. S. Army, 1901.

The fascine mattress used in the upper portion of the river is shown in Figs. 110 and 111. In the middle portion lumber and woven mattresses have each been largely used. The woven mattress is formed of a single thickness of brush, usually of willows, woven on poles and fastened together by wire, nails, cables, etc. The width is usually 120 feet, except where special scour has to be met, and they have been made in lengths as great as 10,000 feet. They are sunk by being weighted with stone, and the protection of the upper portion of the bank is secured by grading and riprapping. (Pl. 20.) Usually the river is allowed to do much of this grading, and sometimes two or three seasons elapse after the placing of the mattress before it becomes advisable to riprap the slope above.

Bank protection along the lower portion of the river, on account of the size and velocity of the stream, has been a problem of much difficulty. The general outline of caving bank to be dealt with consists of a steep and often vertical face from 30 to 40 feet high above low water. Below water the constantly falling material maintains a




General Section. Plan of Mattress.

FIG. 110.—Protection Used on the Upper Mississippi River.

slope of 1 to 1 or sometimes flatter, and this gradually tapers off, depths of 35 to 50 feet being found about 100 feet from the bank, and the greatest depths, which usually range from 60 to 100 feet, at 250 to 350 feet from the bank. Much of the caving is due to sand seams (Fig. 97, p. 180), cases being met with where a bank several miles in length will settle vertically for a few feet, and continue settling gradually perhaps for some years and until all the loose material has been carried away, when a new settlement begins.

The first work was done at Memphis in 1877, when mattresses of willow poles were made in widths of 50 to 60 feet, and in lengths of 120 to 130 feet.* They were sunk separately, the ends being made to lap 10 or 15 feet, and though temporarily successful, they proved of short life, as owing to their small width they soon became undermined and slipped down the bank. In consequence of this experience the size of the mattresses in work elsewhere was subsequently increased, and in 1886 they were made 250 feet wide and 600 feet long. Riprap was used to sink them, and gravel in sacks was also tried, but the sacks soon rotted and the gravel was carried away by the currents. At the present

* Information as to work on this portion of the river is derived chiefly from an unpublished report by Major E. E. Winslow, Corps of Engineers, U.S.A., compiled about 1900, and reproduced by his permission.

FIG. 111.—Models of Shore Protection, Pile Dike, and Plant. Improvement of the Mississippi River between the Mouths of Missouri and Ohio Rivers.

time the usual dimensions are 300 feet wide and 1000 feet long. In much of the earlier work there was a good deal of loss due to drift getting under the mattresses during construction; to inexperience in handling; to the insufficient width of the mattresses, which became undermined in consequence; to seepage causing sliding of the banks; and to the too open nature of the construction, which allowed the eddies to work between the interstices and undermine portions of the mattresses which then broke under the weight of the riprap. This last cause usually limited the life of a mattress to about ten years. Losses also occurred from the mattresses gradually drawing down the bank as they settled in place, leaving a bare space along the upper edges; this was quickly attacked by the current, and unless promptly checked it would lead to the undermining of the work from behind. The upper and lower ends were also exposed to danger when not sufficiently protected. A combination of these causes at Osceola Bar in 1890 eroded the bank so greatly that a large mattress was flanked and left 500 feet out in the river and had to be removed, as it became a danger to navigation. An example of the risks from drift was met with at this same locality in that year. A large mat was being sunk when a rise occurred, bringing down much drift. Barges with hanging chains were moored above and were successful in intercepting the débris, but before the mat was safe on the bottom the drift had piled up under the outer barge until it was lifted almost entirely out of the water.

The bank protection above water was at this time made of loose brush lightly weighted with stone, this type being employed as it was believed that the brush would fill with sediment and make a permanent surface. The currents, however, were too strong to allow any such deposits to form, and the brush being thus exposed soon rotted. The grade of the banks was also made somewhat too steep, the flattest used being 1 vertical to $2\frac{1}{2}$ horizontal, whereas the natural slopes formed by the river were about 1 to 5. As a consequence the current sometimes rolled the riprap off the bank. Slopes of 1 vertical to 4 horizontal were subsequently used, but as this proved too costly, the grade of 1 to 3 was finally adopted. Later on mattresses of brush were used on the upper banks, but these were equally subject to decay, and in addition they occasionally slipped down the bank and folded up or buckled. In 1891 brush above water was discarded entirely and riprap substituted, the best results being obtained where the pieces were laid on 3 or 4 inches of spalls as a protection against wave action.

The danger of the pushing out of the subaqueous protection by seepage is now guarded against successfully, where the water is not too deep, by driving two rows of piling about 12 feet apart along the toe of the bank. This piling is composed of sheet piles supported by walings and round piles, the latter being about 8 feet centers. These are braced crosswise and every other bent is tied back to the bank by a wire cable, and the space between the rows of sheet piling is filled with brush mats weighted with stone. Similar brush and stone is placed behind and the whole serves as a mass to intercept the material flowing from the bank.

The following notes are taken from the report just referred to. Quotations have been made at some length as the principles described and the experiences developed are applicable to a greater or less extent to all alluvial rivers. It is to be noted that the report describes conditions along the lower Mississippi, the most difficult portion of the river.

" Beginning over twenty years ago (between 1870 and 1880) the work has progressed with varying success; alterations have been made from time to time as experience showed them to be desirable, until there was at length evolved a successful type of work that seems suited to the conditions met with. During this period a number of different types of work have been tried and much experience has been gained, and it will perhaps be well to state briefly the most important conclusions.

" The object of bank revetment is to protect the banks of the river from destruction by the currents. The active agency in this bank destruction is erosion, and bank revetment must therefore prevent this erosion. Perhaps the easiest and most obvious method of doing this is by laying over the exposed surface of the erodible bank a non-erodible covering, and this is the method adopted. This non-erodible covering must of course be fairly durable, must have sufficient strength to withstand any strain put upon it, and must be free from interstices through which scour might take place.

" Owing to the fluctuations in the river, the different parts of the bank are subjected to different conditions. Below the low-water line the bank is always wet, above that line sometimes wet and sometimes dry, and this difference in conditions allows, if it does not require, that the portion of the bank above and below low water be treated differently.

" Above low water the bank can be seen and the non-erodible covering can be laid with ease, and it has been found that here a properly laid stone pavement forms an efficient protection. The individual stones of this paving must of course be so large that they cannot be moved by the current, and it was found by experience that stones of this size are not of themselves sufficient, as through their interstices scour could take place when the bank was sandy and the current strong. This has been prevented by first covering the bank with a layer of spalls or crushed rock, of such thickness and closeness as to prevent scour through them, and to hold this down with a layer of larger stone. Such a protection, when carefully laid, has been proven to be all that is required for the upper bank.

" Below low water the conditions are different. As it is impossible to construct the non-erodible covering actually in place, the next best thing is done, constructing it on the surface of the water above where it is to lie, and when constructed sinking it. In this way the continuity of the covering can be assured, and it can be placed exactly where it is needed. As has been stated, this covering must not contain interstices through which scour can take place, for in such a case not only would the purpose of the covering not be fulfilled, but by such scour its own eventual destruction would be assured.

" The covering actually used has been a mattress of brush, and the only kind that

has been found to prevent scour in very rapid currents is the fascine mattress. (Pl. 21.) In moderate currents another type of mattress might do, but as changes in current conditions are always likely to happen, the fascine mattress only should be used.*

"As the immediate object of bank revetment is to prevent erosion it must cover all parts of the bank where erosion can take place, or at least where its effects are at all serious. As at one stage or another erosion may take place at any point between the top of the bank and the foot of the subaqueous slope, it is necessary, in order to stop all erosion, that the revetment work should cover the entire bank from its top to the deepest water.

"In the work that has been done the paving has only in a few cases been carried to the top of the bank; in the early works because it was not considered necessary, and since 1897 because of the insufficiency of funds. The Hopefield experience † of 1897 showed the absolute necessity of such an extension in very exposed places, and as every place where revetment is needed may be at some time subject to severe attack it is safest, and usually cheapest in the long run, to extend the paving to the top in the beginning.

"The subaqueous mat should in general, and where possible, be extended to the deepest water, for out to that line erosion has recently been active even if it be not now actually going on, but if the subaqueous slope is gentle it may not be practicable to extend the revetment to the greatest depth. Generally this point can be reached with mats 300 feet from the low-water line, and in cases where the deepest water lies further out, the bottom is at that distance usually quite flat and such cases are not found along the most rapidly caving banks. But even if the mat be carried to or beyond the greatest depths, some scouring along the outer edge may be expected. If the bank be a rapidly caving one, and such banks are most in need of revetment, the growth of the bar opposite will probably be somewhat behind the caving, and such a bar will continue to grow at least for some years after the construction of the revetment. The increase of cross-section by the caving of the bank being stopped, the decrease from bar growth will probably be felt in an increased scouring action along the bottom, that is, along the outside edge of the mattress. To prevent damage from this cause the mattress must be so flexible that it can follow down this scour as it occurs, without being itself injured. Action of this kind cannot proceed far, and if the mattress be wide enough so that its outer edge can reach to a sufficient depth without the slope becoming too steep, all that is required has been accomplished, and it would appear that for this purpose mats reaching 300 feet or so from the low-water line are of ample width, and in many localities narrower mats will suffice. Of course future experience may show that in some places still wider mats are needed, but there is nothing in the construction of the fascine mattress to

* The ordinary mattresses, composed of woven brush or of poles laid side by side, were found to be too open, and were gradually undermined in many cases by the currents working through the interstices.

† The work at this locality is referred to in the paragraphs on Cut-offs, p. 235 and after.

prevent its being made of any width desired, by merely using additional mooring and mat barges, and more and possibly stronger mooring cables.

"*Bank Sloughing*.—As regards sliding or sloughing, the river bank is like any other mass of earth, there being an angle of steepness beyond which the bank is in an unstable condition and liable, upon being disturbed, to slide. This angle varies greatly with the character of the bank and its degree of saturation, and depends also upon whether or not it has recently been disturbed, and its value in any case can only be determined by experiment. As the river banks are subject to overflow, and at such times become saturated, all plans for insuring their permanency must be based on the assumption of complete saturation.

"Beyond the angle of repose the bank can only be permanently held by some form of retaining wall, and as a retaining wall a revetment cannot ordinarily be expected to act, and hence the sloughing or sliding of the bank can only be prevented by grading the bank before revetment to a proper slope. As far as the part of the bank above low water is concerned this grading is an easy matter, but should the bank below the low-water line be too steep no method for grading it has been as yet tried in these districts. Various methods of grading by scraping have been suggested, but so far as is known they have not been used with success.

"Experience on this river both in bank revetment and levee construction has shown that, in general, if the upper bank be cut back to a slope of 1 vertical to about 3 horizontal, it will be stable even when saturated, though in special cases the soil may be such as to require a more gentle slope. Below low water the slope may be steeper; some banks have been successfully held where for several feet the slope was as steep as 1 to $1\frac{1}{2}$, while the depth 100 feet from the shore was as great as 45 feet.

"In revetting steep subaqueous slopes the best method would seem to be to grade the upper bank well back, and thus remove its weight, and if the bank be not already cracked or shattered it may be revetted with probable success. If, however, the bank be already cracked and shattered, further settlement may be expected to take place, and such settlement will probably do more or less damage to the revetment, but the revetment will, where still intact, prevent erosion; the settlement must in time cease and by constant watchfulness and quickly repairing any damage done, the bank can probably be held in spite of its shattered condition, unless there be complications due to seepage.

"*Seepage*.—A large amount of the trouble with banks can be laid to this cause. The banks of the river are in general built up of layers of clay, sand and gravel. The clays are generally impervious, while the sand and gravel strata are quite pervious, and when the river rises, as in floods, there is a heavy water pressure against the bank and the pervious strata become saturated. Then as the river falls the pressure against the bank decreases, the water in the pervious strata returns to the river, and as it does this it tends to take with it the material of the pervious stratum, and if this material be fine sand, or what is known as quicksand, some of it will flow out from near the bank surface,

its outcrop will assume a much flatter slope, thus partially undermining the overlying strata and allowing them to break off and fall in.

"When the pervious stratum outcrops above low water the trouble is not so bad and can usually be remedied with comparative ease, but when the stratum lies below the surface, and material is very fine, considerable trouble may be expected. The mattress laid on the exposed face of this stratum of course will diminish the outflow of the material, but if the sand be fine enough, the returning water will be able to carry it through even a fascine mattress, and a settlement or sloughing of the bank will eventually take place in spite of the mattress. This slide will probably do some damage to the revetment, but enough of the work will still probably be left to retard, if it does not prevent, erosion. However, the material brought down by the slide will probably cover the face of the fine stratum and stop the outflow through it, and a restoration of the revetment should then hold the bank. Further settlement may of course go on for some time, but it will probably be slight and with care and quick repairs the bank can probably be held.

"This trouble from seepage is of course intensified by the existence of lakes or streams having a connection with the fine and pervious stratum.

"*Types of Revetment.*—As far as its continuity is concerned three types of revetment have been tried, the continuous type, the interrupted method, and dikes; the first alone with success.

"The continuous type needs little comment. If properly constructed it must prove successful. In general, the more regular the bank along which it is placed the easier and cheaper will its construction be, and the less it will cost to maintain. Every projection in such a bank is not only of itself subject to a more severe attack and therefore more liable to give way, but by the whirls and eddies it causes it disturbs the regularity of the river regimen, and may threaten the work just below it.

"Above low water such projecting points can, by grading, be easily removed, or at least their saliency can be reduced, but below the water surface such grading is not practical, and where such irregularities exist they must, in general, be incorporated in the revetment, special care being taken with the rock in their vicinity.

"In this connection one point is important: as the end of any revetment left exposed during high water is certain to be left by caving on a projecting point, it is desirable, when a long and regular bank is to be revetted, that as much as possible be revetted in one season and in one continuous piece without gaps.

"The interrupted method aims to reduce the cost of the work by leaving gaps in the revetment. It was tried but once, at Fletcher's Bend in 1888. Here the attack of the current was quite light for some years, so that no very great caving took place in the gaps, but when the current became stronger these pockets enlarged rapidly, destroying the exposed ends of the revetment, and large stone groins had to be constructed to prevent further damage, and it is believed that had the current been as strong here as at

other places that even these groins would have proved insufficient. Indeed, judging from the experience in other places and the rapidity with which small faults are enlarged when the current attack is very strong, no work of the interrupted type will hold unless the gaps be very small and the protecting works be very large, and in this case the advantage sought in this kind of work, cheapness, would be lost.

" The experience on this river with dikes used as bank revetment has shown them not to be suited to that purpose. In every case where a single dike has been built so as to make a projection from the bank, it has not only failed to protect the bank below it but has, by its eddy, caused or accelerated bank destruction. Where dikes have been used in sets and have been subjected to severe attack, they have failed until additional work had been placed in the intervals and the set of dikes converted into practically a continuous revetment.

" One of the results expected from the use of dikes was the causing of deposit between them and the building out of the bank. This they have clearly failed to do and the gaps between the dikes have not been filled up, but have been cut into the bank. Were the gaps made very small and the dikes very large, they might possibly serve, but at the expense of economy, and, in general, experience shows clearly that when the caving bank is fairly regular it is both better and cheaper to use continuous revetment.

" In certain cases, however, dikes have been found very useful, namely as eddy breakers in pockets. The formation of eddies in pockets and the caving due to them, have been fully explained.* By building a dike near the center of an eddy pocket, the eddy is broken in two, its action interfered with, the velocity of the eddy current is greatly reduced, and, if the dike be properly planned, scouring is stopped and a deposit is caused. In pockets and similar localities many dikes have been built and with about uniform success. However, if the pocket be large and the eddy in it strong, it is advisable to first floor the the pocket completely over with a mattress and then build the dike, and in addition a portion of the sides of the pocket can with advantage be paved. Indeed, at Hopefield Bend it became necessary to use a dike to break up an eddy in a pocket that had been completely revetted, the eddy current being so strong as to make the revetment of the pocket difficult to maintain.

" As a final conclusion, it may be said that it has been clearly shown that for use on this river only the continuous type of work should in general be used; that such continuous work be constructed of the fascine subaqueous mattress of proper width, with an upper bank paving of stone and spalls on a properly graded slope, or of some equivalent construction. With this type of work and with proper care in its construction and maintenance, it is believed that any of the banks of the river can be successfully protected.

" The materials used in this revetment are brush and stone, of which the supply has so far been ample. Some years ago it was feared that the supply of brush would prove insufficient if much work had to be done, but from appearances at present it would

* See p. 21.

seem that the work will always be stopped by exhaustion of funds long before the brush is used up. Other material may of course be substituted, and in fact experiments have been made in the Fourth District with concrete as a substitute for rock and rough timber for brush. (See pp. 187 and 192 respectively.)

"*Construction of the Present Standard Type of Bank Revetment.—Plant.*—The floating plant needed by a revetment party consists of mooring barges, mat barges, fascine barges, brush and stone barges, drift fender barges (needed at times), hydraulic grader and pile driver, towboats, quarter and store boats, etc. Ordinary decked barges are used; they are strengthened where required by bolting their sides together with numerous iron rods and adding the necessary capstans, launching ways, timber heads for attaching the lines and cables, etc. The barges in use are about 120 feet long by 30 feet wide, two or three of them being placed end to end, so that the combined length will be fully equal to the width of the mattress.

"To keep drift from getting under the mattress during construction it is the usual practice to hang chains, arranged in festoons, from the upstream side of the mooring barges. Two chains are used, extending along the barges and drawn up with rope straps and fastened to alternate timber heads. These chains hang down a few feet below the bottom of the barges and by crossing each other present a good barrier to catch the floating drift. The chain used is generally of $\frac{1}{2}$-inch diameter iron. (Fig. *a*, Pl. 21.) Where much drift is running it is sometimes expedient to anchor a set of barges some distance above the mooring barges for the sole purpose of catching the drift flow. These barges have their own mooring ropes and drift-catching chains, and extend far enough out into the river to thoroughly shield the mattress below. When the drift barges are used the mooring cables from the mat are anchored to them and a straight up- and downstream lead is thus obtained. This is of considerable advantage in controlling the mat while being sunk.

"The grader is simply a boat containing the pumps necessary for the hydraulic grading. Pumps of various designs have been employed, and in general, to be efficient, the machine must be able to supply at least two streams discharging through 150 feet of 3-inch hose and 1-inch nozzles, maintaining a pump pressure of about 160 pounds per square inch. As one cluster of piles is usually necessary as an abutment for the mooring barges, it has been found convenient to have one end of the grader made into a pile driver. This makes the mat plant independent and is a convenient arrangement, as the pumps can furnish the jets for the pile driver.

In addition to the foregoing there are used towboats, quarter and store boats, etc.

"*Cables, Mooring Lines, etc.*—For mooring the barges and the mattress, crucible cast steel wire ropes are now exclusively used. The sizes of these vary from 600 feet to 1000 feet in length and from $\frac{3}{4}$ inch to $1\frac{1}{8}$ inch in diameter. The number used depends upon the width of the mat and strength of the current. The longer and larger are used along the outside of both barges and mats, as here the strain is greatest, and it requires

longer ropes to secure proper fastenings to the bank. For constructing a mattress 300 feet wide in a strong current, say with a velocity along its outside of about 6 feet per second, it is usual to anchor the mooring barges with 8 ropes. The three on the outside having diameters of $1\frac{1}{8}$ inches, the three along the middle of 1 inch diameter, and the two along the inside of $\frac{3}{4}$ inch diameter. (Pl. 23.)

"To the mattress there should be attached three $1\frac{1}{8}$-inch ropes, three 1-inch and two $\frac{3}{4}$-inch ropes, arranged with the larger on the outside. The lines to the mooring barges are fastened to the large timber heads of these barges. Those to the mattress pass under the mooring barges and are fastened to the mathead by means of strong chains wrapped around it. In addition to these main fastenings, manila ropes, 1 and $1\frac{1}{4}$ inches in diameter, are run from timber heads on the lower side of the mooring barges around the mat head and back again to the same timber head; these are known as slip lines, and by slacking off one end of them the mat can be easily lowered while being sunk. To control the mat barges 2-inch manila ropes are run from each end of them to the mooring barges, and as successive sections of the mat are constructed, they are launched into the river by slacking these ropes.

"*Material used in Construction.*—The materials used consist of brush, poles, hardwood poles, stone, spalls, wire strand, staples, clips, etc.

"*Brush.*—This should be straight, green, freshly cut willows, and generally not more than 4 nor less than 1 inch in diameter at the butts. For the core of the fascine, however, a small quantity of larger brush can be used advantageously, it being that found the larger brush, containing more heart, does not so rapidly deteriorate under water.

"*Poles.*—These should be not less than 3 inches in diameter at the small end nor more than 6 inches in diameter at the butts, and not less than 24 feet long and of willow or cottonwood.

"*Hardwood Poles.*—These are usually of sycamore or ash, of sizes up to 8-inch butt diameter. They are used in the mat head only, to prevent the strap chains from cutting through them, as might happen with a softer wood.

"The brush and poles are cut fresh on the river bank and as close to the work as possible, and for all work done so far there has been an ample supply obtainable within easy towage.

"*Stone and Spalls.*—The requirements for riprap are that the stone shall be sound and durable, and in pieces weighing between 10 and 35 pounds. Pieces of the same stone weighing between 1 ounce and 10 pounds are classed as quarry spalls. Most of that used has been limestone.

"There being no stone obtainable along this river, it has to be brought from a distance. A part of the supply has been boated down from quarries in the Ohio and upper Mississippi rivers, 75 miles or farther above Cairo, and part has been received by rail from points on the Kansas City Railroad, 100 miles or so from the river. This long transportation increases the cost of stone.

"*Wire Strand, etc.*—The wire and wire strands used are the ordinary commercial product, made of good annealed steel wire fairly well galvanized. The $\frac{1}{4}$-inch and $\frac{5}{16}$-inch strands are composed of seven wires each, while the $\frac{1}{2}$-inch diameter strands are made of 19 wires to give them more flexibility.

"*Clearing.*—Usually the first thing to be done is to clear the bank, removing from the slope all fallen trees, stumps and snags, and from the top of the bank the trees and brush back to slightly beyond the top line of the slope to be graded. When there are snags below low water, as is frequently the case, these must be removed; generally a towboat for pulling them and the use of dynamite to loosen them will suffice; but when they are too firmly embedded in the earth it has been found at times necessary to secure the services of one of the regular United States snagboats operating along this portion of the river.

"*Preparing Anchorages.*—The mooring cables from the barges and mat are fastened to trees or stumps, and when these cannot be found at convenient points, to "dead men." These dead men are heavy logs buried at least 4 feet in the earth with their lengths normal to the direction of pull. By using logs of 20 or more feet in length, two or more lines can be fastened to each dead man.

"*Driving Pile Abutment.*—This consists of a single pile, or of a cluster of piles, depending on the strength of the current, driven a few feet inside of the water line and a short distance above the location for the head of the mat. This pile or cluster is braced against the bank and serves to hold the mooring barges in place during mat construction and sinking. After its usefulness has passed the piles are sawed off and removed.

"*Placing Mooring Barges.*—The abutment having been driven and the anchorages prepared, the mooring barges are brought up and are temporarily placed along the shore with their downstream end just above the abutment. While in this position the barges are firmly lashed together end to end, so that their total length is greater than the desired width of the mattress. When all is ready the barges are swung out and the cables are adjusted so as to hold them in their position, normal to the bank and with their shore end resting against the pile abutment. (Pl. 23.)

"When the mooring barges have been placed the mattress barges are brought up on their downstream side, with the lower end of the ways nearly touching the mooring barges. They are held in this position by lines from the inner and outer ends of the mat barges to the mooring barges. Next below the mat barges are placed the fascine barges, and below them barges of brush, which are renewed as fast as emptied.

"*Construction* (Pls. 21, 22, and 23.)—The plant being in place, the first step is to construct the mat head. This is built across the ways and consists of hardwood poles, formed into a fascine of about 2 feet diameter, and as long as the mat is wide. It is held together by being well bound with numerous ties of wire strand. To the mat head are now attached the wire strands leading from each drum, each strand passing under the mat head, twice around it, its end being fastened with two wire rope clips. These

Weaving a Fascine Mattress on the Lower Mississippi River.

Woven Mattress under Construction.

Woven Mattress just Beginning to Sink.

Fascine Mattress, Completed. 300×1125 feet.

strands are of $\frac{5}{16}$ inch diameter, except the two on the outside, which are $\frac{1}{2}$ inch in diameter. (Pl. 23.)

"These strands are known as the 'bottom' strands of the mat and are continuous throughout its length and give it most of its longitudinal strength. They are, for construction, wound on the drums, pass thence up and over sheaves at the upper end of the ways, down the ways to the mat, and under the mat to its head.

"When all these bottom strands have been adjusted, work is begun on the first fascine. The fascine is built in the 'formers,' the brush being placed so as to break joints and give the fascine a nearly uniform diameter of 12 inches throughout its entire length, which is equal to the width of the mat. When sufficient brush is placed in the formers it is tied together with rope yarn at sufficient intervals to hold it in shape. It is then pushed out of the formers by the levers and slides down against the mat head, where it is taken in charge by the 'sewing' gang, who securely tie it into the mat at each bottom strand with other strands of $\frac{1}{4}$ inch diameter, called 'sewing' strands. The latter are used in lengths of about 60 feet, this being the greatest length that it is convenient to handle, and when one length is used up another is spliced on. The sewing strands are first tied to the mat head, then they pass over and around the fascine, crossing the bottom strand and up next to the head, and are then hauled tight with tackle blocks and lines, and secured by a staple from slipping, when the blocks are released. While the strands are being hauled taut the compacting of the fascine is assisted by the blows of a wooden maul. The sewing gang proceeds from one strand to the others, repeating the operation, and by the time one fascine is sewed in the next is ready for them.

"In the manner just described the succeeding fascines are made and sewed into the mat. This method not only compresses the brush in the fascine very tightly, but joins the fascines closely together. The tackles used consists of two 6-inch double blocks with $\frac{3}{4}$-inch fall lines. One end of the tackle is lashed to the mat already built or, in first starting, to the ways, and to the other end is attached a cam-shaped clamp, which will grip the strand wherever placed. The staples are ordinary fence staples, driven around the strand and into the brush. They serve only the temporary purpose of holding the strain on the strand when the blocks are released, as they are pulled out when the next fascine is added.

"At intervals of about 10 feet the sewing and bottom strands are securely fastened together, clips being used for the purpose.

"When several fascines have been sewed in, the whole construction is pushed down to the lower end of the ways, the mat head coming against the mooring barges and the slip lines and mooring cables are attached. The slip lines hold the mat to the mooring barge during construction and serve, when slowly paid out, to guide the mat in sinking. Before the slip lines and cables are attached, the mat head and its adjoining fascine are forced apart by means of wedges so as to leave sufficient space for the ropes and chain

to pass through. These wedges are allowed to remain and thus permit these ropes and chains to be easily withdrawn after the mat is sunk.

"While the moorings are being attached, the construction of the mattress is continued, and when enough has been built to nearly fill the ways a launch is made. The lines joining the mat and mooring barges are slackened, the brakes on the cable drums are released, and the current pushes the mat barges downstream, the mat sliding off into the water, the motion being readily controlled by the proper handling of the lines and brakes. When the barges have dropped so far that only a few feet of the mat remain on the ways the motion is stopped, the lines and brakes fastened, and mat construction can be resumed. After the launch has been made all the bottom strands should be taut and have a uniform strain.

"The construction of the mattress is thus continued, launch after launch being made as the ways are filled until the desired length of mattress has been built, when the bottom strands are cut off and tied around the last fascine, and then the final launch is made completely off the ways and the mat barges are then removed.

"As soon as about 100 feet of mattress have been launched on the river, the work of cabling and poling it is begun, these operations being continued so as to keep closely up with the construction.

"*Cabling.*—The strength that the mat has in a longitudinal direction is practically the strength of the bottom wire strands, for the sewing strands frequently have some little slack in them at places. As this strength may be insufficient to prevent the mat from tearing while being sunk in currents of considerable velocity, it must be increased to render the sinking safe. Hence it is the rule to add more wire strands, the number and size being dependent upon the strength of the current. One-half inch wire strands are always attached to the head where the mooring lines are secured, and run along the mat for at least 100 feet ordinarily, and for a greater length when the current is stronger, and sometimes for the entire length of the mat. When the current is extremely strong, intermediate strands are placed in the same manner, and there appears to be no reason why the tensile strength of a mat cannot be made so strong that failure can only occur by the breaking of its mooring cables. These extra strands or cables are placed on top of the mat, the process of placing them being called "cabling." All top cables should, as far as possible, be equally strained and this strain should be equal to that of the bottom strand.

"*Poling.*—This consists in placing longitudinal lines of willow poles on top of the mat at intervals of 8 feet, this being done after the adjustment of the top cables. These poles are lap-spliced with one strong wire tie, and tied to the fascines with wire at every 4 to 5 feet, the alternate tie wires being of one piece of No. 9 gauge silicon bronze wire, the object of using this being that it will be effective after the other ties, which are of galvanized steel wire, have lost their strength by corrosion. On top of these longitudinal poles similar poles are placed, crossing the mat in a transverse direction, and wired to the

longitudinals at the intersections; these poles are spaced about 8 feet apart from the mat head for a short distance, and then 16 feet apart down to about 100 feet from the head, below which they are omitted except along the outstream edge, where transverse poles covering three longitudinals, spaced about 8 feet apart, are placed for the entire length of the mat, and on top of these again are placed two or three lines of longitudinal poles. The object of the poling system is two-fold; first, to add the requisite stiffness of the mat in a longitudinal direction, and second, to prevent too much shifting of the stone ballast in sinking, the head and the outside edge being especially cribbed up for the latter purpose.

" The mat as thus completed has its longitudinal strength furnished by the top and bottom strands, which can of course be increased to any desired extent. Its transverse strength is that of the fascines only, and due to the tightness with which they are bound. This has been found ample for all needs, but, if necessary, this could be increased by wire strand laid across the mat at intervals. The mat is sufficiently flexible to adapt itself to the general irregularities of the bottom that exist at the time of sinking, or that may develop later. From its construction it is evident that it is quite dense, and, if properly constructed, contains no holes through which scour can take place, and being non-erodible itself it will, when sunk, furnish a non-erodible covering for the portion of bank which it covers. The mat described is of 300 feet width, but there is nothing in its construction to prevent that width from being largely increased, should experience show it to be desirable.

" The completion of the poling of a section of mat makes it ready for ballasting, but in practice this is never begun until a few days before the completion of the mat. This river carries enough sediment to quickly silt up a fascine when submerged, and this silting up and the absorption of water by the brush, decrease the buoyancy to such an extent that the mat tends to sink when ballasted. Therefore ballasting should not be begun until the mat is within three or at most four days of completion. In several cases where, owing to lack of stone sufficient to sink, the ballasted mat has lain for four or more days on the surface, it began to sag along the middle, this sag in one case amounting to as much as 15 feet. In such a case there is some danger of rupture and distortion.

" When ready for ballasting, barges loaded with stone are placed along the outer edge of the mat, planks are run out from them to and across the mat, then stone is carried out by wheelbarrows, distributing the weight on the mat uniformly until it is barely afloat. The ballasting is begun at the head and continued downstream, the barges being dropped down along the mat as each section is ballasted. A considerable quantity of stone is also placed on the mooring barges along their downstream edge for use when the mat is being lowered at the commencement of the sinking. The amount of ballast used is ordinarily about a third of a cubic yard to 100 square feet of mat.

" *Sinking* (Pl. 20.)—After the ballasting is completed, the mat is pulled close in to the shore and held there by wire strands to fastenings on the bank. Loaded stone

barges, lashed end to end, having a combined length sufficient to cover the width of the mat, are brought to and are hung from the mooring barges along the upper outside of the mat. Stone from the mooring barge is cast onto the mat near its head, and by slacking the slip lines the head of the mat is allowed to sink a few feet. The stone barges are then pulled in over the mat alongside the mooring barges, and long lines are passed from the mooring barges to the stone barges to control the movements of the latter. When all preparations have been completed, a large force of men is placed along both sides of the stone barges, considerable stone is cast on the mat, the slip lines are slackened, and the mat is gradually lowered to the bottom. The drop lines to the stone barge are then paid out, and the stone barge is thus permitted to drift downstream over the mattress, the laborers throwing off stone from all sides as rapidly as possible, this being continued until the mat is completely sunk. Under favorable conditions the time of sinking is less than an hour for a thousand feet mat, counting from the time of sinking the mathead. To facilitate the sinking, it is usual to have a steamer attached to the outer end of the stone barge, and a line to men on shore from the inner end, to control the barge from swinging too far in either direction. After dropping the stone barge over the entire length of the mat and sinking the latter, it is usual to bring it back to the mooring barge and repeat the operation, this time permitting it to drift slowly over the mat, and casting off the stone so as to secure as uniform a distribution as practicable and insuring the mat being well down.

" It sometimes happens that a mat is sunk in a locality where the current is reversed. In such a case provision against the mat's ' crawling ' upstream must be made by anchoring the lower edge to the bank below with one or more cables, according to the strength of the eddy. It is very seldom, however, that this reverse current extends outstream more than half the width of the mat, therefore only such part of the mat as will be subjected to the eddy need be anchored, but this precaution should never be neglected. When the mat is supposed to be safely sunk, the mooring lines are slackened, the shackle pins pulled up and all chains and cables are recovered and the mooring barges removed.

" The total quantity of stone needed to ballast and sink a mat is ordinarily $\frac{3}{4}$ of a cubic yard to 100 square feet of mat.

" *Rapidity of Work.*—With a full and well-trained force and under average conditions, about 120 feet of mat can be constructed each day, and under favorable conditions a mat 1000 feet long should be built and sunk every two weeks.

" When a bank is straight the mat is made so as to lap on the shore a few feet above the water. If the shore line be gently curved the mat can be made to fit it by sliding off one end of the mat from the barge more than the other and making the fascines conical instead of cylindrical and thus make the mat curved.

" It frequently happens, however, that some of the indentations of the shore are so deep in comparison to the width, that the main mats cannot be well fitted to them. In such cases recourse must be had to connecting mats. These mats are built similar

to the main one, except that the fascines lie parallel to the shore instead of normal to it and that no top cables are used. It is built out by successive launches until it overlaps the main mat by 20 feet, when it is ballasted and sunk. Before this is done the mat is anchored to the shore about every 50 feet by wire strands fastened to stumps or trees. Their principal duty is to prevent sliding down of the mat while it is being sunk.

"After the sinking of all the subaqueous mats work is begun on the upper bank, and before the paving can be laid on this it must be first cut to a proper slope.

"*Grading Upper Bank.*—The slope to which the upper bank is graded is usually one vertical to three horizontal, though in bad soils and on exposed salients this slope is made more gentle. If, as sometimes happens, the bank is already at or near that slope and merely needs a little smoothing off, this can be done with shovels, but in general the hydraulic method costs less. Two streams are generally used. The nozzle for each hose pipe is fastened by a universal joint to a piece of iron pipe driven into the ground and is operated by a long wooden lever. This holds the stream steady and permits it to be thrown in any necessary direction. The nozzles are held quite close to the face of the cut and for protection from the spray the men are furnished with waterproof clothing. One stream is used to undercut the bank and the other to wash the fallen material into the river. The usual practice is to work downstream, cutting the top of the bank well in advance, so that the wash from this will flow as closely as practicable along the face of the cut. In this way a uniform grade can be most easily obtained, but with some banks composed of more or less sand it is practically impossible, however much care be taken, to prevent the washing out of gullies, which must of course be subsequently filled up. Generally it is necessary after grading to do some little trimming with shovels, but the amount of this should be made as small as possible, as a bank is better able to stand the attack of the current the less it has been disturbed. The principal obstacles to rapid work are stumps and buried logs. All stumps not washed out by the grading must be afterwards removed by chopping down flush with the slope; otherwise, when they become submerged, small eddies will be created behind each one, which may damage the paving.

"It was formerly the custom to grade the bank before building the subaqueous mat, as it was thought that the material washed down would make a more regular subaqueous slope. This was subsequently abandoned as a rule, but it can, if desired, be used in future at a place where there is no danger of caving going on during mat construction and thus destroying the slope already graded. By waiting to grade until after the mats are sunk, the subaqueous slope is held, the inshore edge of the mat rests on firm and undisturbed material, and the material washed down on the mattress helps to silt it up, hold it in place and fills up any cavities beneath it.

"*Paving.*—After the bank has been graded paving is begun. There is first spread over the bank a layer about 3 or 4 inches thick of quarry spalls or crushed rock. Upon this a carefully pitched layer of riprap with a thickness of from 6 to 8 inches is placed.

Care is taken to leave as small voids as possible, and the top surface is left rough. This makes the total thickness of the paving from 8 to 12 inches, tapering in thickness from the bottom to the top of the bank. Along the edge of the mat work, and in order to cover any narrow belt left bare by the settlement of the mattress, there is laid quite a heavy ridge of stone 18 inches or more in thickness.

"The bank revetment completed, as above described, covers the bank from the top of the paving to the foot of the mattress with a covering non-erodible itself, and containing no holes through which scour can take place, and erosion can, as long as the work remains intact, take place only above the paving or outside the mattress, and by a proper extension of paving and mattress damaging scour at these places can be prevented.

"The weak point is at the junction of the two kinds of work. The mat can always be expected to settle a little after sinking, but the amount of this settlement is ordinarily quite small and soon ceases, and in all probability will have stopped entirely before the paving is laid, and should any slight settlement take place afterwards, the stone ridge will probably fall down into the gap and prevent any damage.

"*Time of Work.*—Bank revetment should always be built at as low a stage as possible, and preferably when the water is falling slowly, not only because the absence of drift and slack current at such a time make the work easy and reduce its cost, but for other reasons. The paving can only be laid on the visible parts of the bank, and it is necessary, therefore, that the mats should lap far enough above the water surface to permit of a proper connection being made, and every foot of upper bank covered with a mattress means that much taken from its outstream reach. Then again all that part of the mat above low water is subject to rapid decay and must eventually be replaced by stone. In practice it is of course impossible to put all the work in at extreme low water, and it is therefore necessary to go over the work during some succeeding low stage and carefully pave over the part of the mat exposed so as to protect the bank even after the brush decays.

"*Repairs.*—The stone of the upper bank work is of course not subject to deterioration while as much of the brush as is always wet is fairly durable, but it cannot be expected that the work can be left to itself absolutely and will need no care or repairs. Bank revetment, like most constructions, may be damaged in many ways, and it is only by constant care and by repairing all faults as soon as they occur that it can be maintained in good condition. The paving just above the low-water line must be watched especially after the first high water. The paving higher up may be injured by boats as they land, pressing holes into the soft banks, by floating logs, and in other ways, and all such holes must be repaired before the action of the current enlarges them. One special cause of damage is concentrated surface drainage, flowing under the paving and cutting gullies into the bank. When not practicable to divert this drainage it must be carried over the slope in specially prepared surface drains. Places where the subaqueous bank, when revetted, is very steep, where the bank is known to be shattered, or where seepage is

expected, must be especially carefully watched, and all faults that may develop must be repaired at once, as it is only by such quick repairs that such a bank can be held.

"*Durability.*—If the revetment after construction be not broken by extraneous causes, its life will depend upon the durability of the material of which it is constructed, especially of the brush and wire of the mattress, as the lasting quality of compact limestone riprap need not be questioned. In the early work brush was used as a covering for the upper bank, but, subject as it was here to alternate submergence in the water and exposure to the atmosphere, it decayed rapidly and possessed little strength after about two seasons, and it was this rapid decay of the brush, permitting it to be easily broken up and exposing the bank to erosion, that led to its disuse and the substitution for it of an all-stone paving. It is commonly thought that brush constantly submerged will not deteriorate, but observations in these districts have shown this not to be true. It does lose its strength, but at a very much slower rate, than when exposed to the air. This rate of decay also varies with the kind and size of the brush, hardwoods deteriorating much less than soft, and the heart of the wood less than the sap.

"Some investigations as to the life of the willow brush have recently been made by taking samples from subaqueous mats which were sunk at different times from six to eighteen years back. The samples of brush from mats sunk six years ago show but little signs of deterioration and the bark still adheres to them in many places. From samples submerged for nine years the bark had disappeared entirely, as have also all small twigs or branches, and the sap of the wood shows considerable deterioration, it being soft and spongy when saturated, and cracking, in the small sizes, in drying. The deterioration of the heart, however, appears to be slight. The brush in general shows a decrease in strength, more marked in the small sizes where the sap predominated. The samples of brush submerged sixteen and eighteen years showed no practical difference in condition. Pieces about $1\frac{1}{2}$ inches in diameter and 15 to 20 feet long could, when grasped at the butt by the hand, be readily broken in two by giving them a sudden jerk. This was done while they were thoroughly saturated with water and were quite heavy. When dried, their strength was largely increased. An examination showed that the sap had so badly deteriorated that it resembled a pulp, and its cohesion, especially when saturated, was practically *nil*, so that the only strength was in the heart, which was small. The larger brush, or that above 2 inches in diameter, still exhibited fair strength, all due to the heart, but this was somewhat softer than brush of half its age of service. The deterioration of the heart was greater in some brush than in others, due probably to the species of the willow, for while in some samples it was almost as firm as the nine-year-old brush, in others it was much softer. A sycamore pole used as a binder, taken from one of the eighteen-year-old mats, had undergone but little change, the fiber being still quite strong and pliable.

"The conclusions drawn are that eighteen-year-old mats are still quite effective as a bank covering, if they are undisturbed, but that they will not, when composed of small

brush, stand bending, and should their outer edge be undermined even to a moderate extent, it is probable that the brush would break up instead of bending to a slope, especially as these mats are of the old woven type with but one thickness of brush. If, however, considerable of the brush is large, the mat might bend somewhat without rupture. Brush in fascines would offer more resistance to breaking, but even in this form, with small brush in condition of the older samples, the resistance would not be great; it would increase, however, materially as the brush increased in size. The investigations show that the sap quickly deteriorates and that the heart possesses considerable durability, and therefore indicates the advisability of using as much large brush as practicable. The durability of a fascine might also be largely increased by using hardwood poles in the core. Even if the deterioration of the heart wood of the willow should continue at the ratio of the past nine years, the brush will serve its purpose for many years to come.

" *Wire.*—Samples of wire and wire strand were taken from the same mats as the brush, and all showed deterioration increasing with their age. Strands of seven wires, only six years old, have had the galvanizing removed from the exposed outside surface, evidently by sand scour, while in the core, where the wires are protected, the galvanizing is still intact. The sections of the wires have also been slightly decreased by corrosion. Where the wire has been in service for a longer period the decrease in both area of section and galvanizing has been greater. This decrease in section is not uniform but in spots, with intervals of wires of nearly full-gauge size between them, varying in length from a few inches to a foot or more in the worst samples, and the galvanizing has not all disappeared even in eighteen-year-old wire. The corrosion at any one place varies in amount up to the complete destruction of the wire. Where a wire in a strand composed of several was found to be badly corroded, most of the other wires adjacent to this bad piece were usually found to be in fair condition, so that the strand still possessed a fair measure of strength.

" The greatest corrosion was found where the wires were tied or twisted together, due probably to the breaking of the galvanizing, which permitted earlier corrosion. While the galvanizing no doubt preserves the wire when intact, it is frequently broken when the wires are tied together by twisting, and may soon be abraded by the moving sand, and when the steel is bared it is possible that corrosion is somewhat accelerated by a chemical or galvanic action setting up between the zinc and steel. Even without such aid the steel will corrode under water, and the manner in which the wire corrodes in spots is somewhat similar to the pitting of the steel plates of the hull of a Mississippi River steamboat.

" Summing up the general condition of the old wire fastenings, it may be said that some are so badly corroded that they are useless, while others, often adjacent, are in fair condition and holding well, and where the ties are composed of a number of wires they are in their present condition more efficient than when the ties are of one or two wires only. The ties as a whole still hold the brush work together and offer a fair margin for

further deterioration, but if their corrosion goes on as rapidly in the future as in the past, it is believed that their efficiency will cease before the efficiency of the brush ceases. It is therefore believed that the employment of silicon bronze wire for tying the top pole grillage to the mat should be continued, and that as large poles as practicable be used for this grillage. The bronze wire will undoubtedly last much longer than any of the other materials, and the corrosion of the clips will probably in nowise affect the durability of the mat.

"Even should the entire efficiency of the wire fastenings be destroyed, the fascines will probably remain in place, because the brush will be quite heavy from water soak, and the spaces between the individual brush will be filled with sediment, so the revetment will still be effective in resisting erosion and would fail by undermining or sloughing. It is probably safe to estimate that a fascine mat built of small brush will be effective for at least twenty-five years, and that when a good proportion of large brush is used its life will be considerably extended."

Details of the cost of the Mississippi protection work will be found at the end of this chapter.

**Construction of Pole or Woven Mattresses.** (Pls. 20 and 23.)—The method of construction of pole mattresses, formerly the only kind used on the lower Mississippi, is as follows:* Hardwood poles, as large as can be conveniently handled by a gang of men and reasonably straight, are laid in two lines on ways over and parallel to the inner gunwale of the construction barge. These poles lap each other 10 to 15 feet, the two lines breaking joints. Where they lap they are spiked together, and they are also tied together with No. 12 galvanized wire at intervals of 10 feet. Two ties are made at the laps. This line of poles is as long as the mattress is wide. About 7 feet 6 inches apart on these poles and at right angles to them the butt ends of weaving-poles, which are of live willow or cottonwood brush from 4 to 6 inches in diameter and 25 to 30 feet long, are fastened with spikes and wire. Another set of poles similar and parallel to the first is placed on these and securely spiked and wired. To facilitate weaving, the tops and the bottoms of the weaving-poles are shaved and the knots trimmed. A cable made of eight strands of No. 12 wire is fastened around the head of the mat at every third weaving-pole and run up alongside of it, the end being fastened thereto by two staples. These cables are 24 feet long, with an eye in one end, to which, after each shift of the mat, a new length is looped in weaving. Ten continuous cables are thus formed in the mat, greatly strengthening it longitudinally. When this head is finished, lines are connected to it from the shore, passing under the mooring barges.

The brush used for weaving is live straight willow of any length over 25 feet and from 2 to 4 inches thick at the butt. The butts are placed over one weaving-pole and project 2 feet beyond, being woven at the other end over the next pole, under the third, over the fourth, and so on, the light ends being always left on top. A strip 5 feet wide

* Annual Report, Chief of Engineers, U. S. Army, 1891, p. 3606.

is thus woven. In the next strip the butts are reversed, the butts changing directions every 5 feet. When the mattress is woven within 2 feet of the end of the poles, giving about 22 feet length of mattress, it is swung in position with the accompanying barges. The head-lines on the barges and mattress are slackened until the barges are nearly normal to the shore, with their inside edge resting against the pile abutment. The slack in the mooring barge cables is now taken in from the bank and the strain equalized. They are then fastened permanently with clamps, as are also the mattress head-lines. An entire shift 22 feet long is then launched, a new set of weaving-poles being spliced to the projecting ends of the first set. This is continued as described to within 2 feet of the top of the second set of poles, when another launch is made, and so on, until the full length of the mattress is obtained.

When three shifts have been launched the construction of a top grillage of framework is begun. This consists of a line of poles laid over and parallel with the weaving-poles, lapping each other, butts to tops, from 6 to 8 feet, and wired to the weaving-poles every 4 feet by lashings 2 feet long, made of two strands of No. 10 wire; transverse poles 8 feet apart for the first 100 feet, and thereafter 16 feet apart, are placed in a similar manner and fastened to the longitudinal ones at the intersections by 2-ft. lashings made of four strands of No. 12 wire.

The purpose of the grillage is to make pens in which the stone is retained, as well as to strengthen the construction. The first set of transverse poles along the inner edge are hardwood, set 8 feet apart throughout the length of the mat, and are used to connect the shore mat, which is also being built, to the river mat.

The construction of the shore mat is as follows: Hardwood poles of the size of the weaving poles are lashed and spiked to the river mattress, and willow or cottonwood poles are spliced to these until they reach up the slope about 40 feet. Alongside and fastened to each of the hardwood poles is a cable made of eight strands of No. 10 wire, one end of which is fastened to two of the adjacent weaving-poles, and the other to the willow poles extended on the slope. Upon the transverse poles are laid longitudinally willow or cottonwood poles 8 feet apart, beginning with the first set about 4 feet from the edge of the mat. The latter poles are wired to the former at their intersections. The longitudinal poles are carried on lines 8 feet apart up to the top of the slope, and on their lower side, 8 feet apart, are driven stakes 2 feet 6 inches above the ground, to the tops of which is loosely fastened a lashing of wire whose bight has first been passed under the pole. These stakes are used down to the pole nearest the water edge. Upon this framework is laid willow brush diagonally with the butts toward the top of the slope and breaking joints throughout. A second layer of brush is put on in the opposite direction, the two thus being at right angles to each other. On top of these layers a second pole framework, fastened similarly to the first, is placed and fastened down firmly by the lashings mentioned above as being tied to the stakes. As fast as the river- and shore-work is finished transverse cables are run across the entire width of the

mat at 16-foot intervals, carried to top of bank, and hauled taut and fastened to trees, stumps, or deadmen placed for the purpose. These cables are fastened to the mat every 16 feet with lashings.

When 400 or 500 feet of river mattress have been completed, longitudinal cables are run out from the mooring barges and securely attached to the mat at 16-foot intervals. One of these is placed close to the outer edge of mat, the others at 30, 37, 38, and 42 feet respectively.

Ballasting can begin after 600 feet have been finished. If the mattresses are not to be more than a thousand feet in length no ballasting need be done until it has all been completed. The stone is wheeled from barges and placed along the transverse poles, loading the entire floating mat until only the poles are above water. By then bringing the stone barges immediately over the mattress and unloading them the structure is sunk to the river-bed. The shore mattress may be ballasted at leisure from barges or otherwise.

Owing to scour along the outer edge many of the mattresses of this type were damaged, and other defects also developed. In order to overcome these difficulties the more flexible and durable fascine mattress, previously described, gradually came into use and is now employed almost entirely.

**Bank Protection on the Missouri River.**—The Missouri, whose characteristics and improvement have been briefly described in the chapter on "Regulation" (p. 89 and after), is a stream of swift current and carries an enormous quantity of sediment. Its channel in consequence is shifting, and the erosion of the banks is often very severe. The prevailing depths, however, are considerably less than on the lower Mississippi. The method of protection in use was the outcome of prolonged study and experience, as the river in its general characteristics was of a type whose improvement had not before been systematically attempted.

Two methods of standard protection are employed, the mattress revetment and the spur-dike. The construction of the latter has been described in the chapter on "Dikes," (p. 164 and after); that of the former is given below:*

"From the nature of the two classes of work revetment is more permanent than dike work. The latter, being largely of wood, partly beneath the water and partly above, must inevitably fall by natural decay in a comparatively short time. Revetment work has no perishable material above water, and is therefore exempt from the processes of decay. Dike work is exposed to the direct onslaught of the river, with its ice and drift and rapid current. The revetment rarely, if ever, receives the attack directly, but at such an oblique angle that it glances off with comparatively little impression. Dike works are avowedly for the purpose of changing the flow of the river; their influence

* Reprinted from the Annual Report, Chief of Engineers, U. S. Army, 1900. See also Paper on the Improvement of the lower Missouri River, by S. Waters Fox (Trans. Am. Soc. C.E., June, 1905), through whose courtesy the accompanying plates are reproduced.

"The pump gets its water supply through a screened suction pipe in a screened well within the hull. The discharge is carried in a 6-inch iron pipe from the pump to both gunwales, at the foot of the masts, for use on either bank of the river. The one not in use is capped. From the boat to the play pipe ashore the water is conducted in 4-inch rubber hose, 75 feet of it being required under ordinary conditions.

"The play pipe at the shore end of the hose is of copper, 4 feet in length, and tapers from 4 inches in diameter at the hose to 2 inches at the nozzle end. It is operated by a wooden lever about 8 feet long, to which it is fitted and strapped. The lever is held against recoil and given full movement in all directions by means of a gimbal which is set at an elevation of 16 to 20 inches above grade in a 2-inch gas pipe that is driven firmly into the earth.

"The jet is played upon the bank at close range. The form of nozzle tip that gives the best results, the most compact solid jet, is cylindrical for 2½ diameter back from the orifice, and thence to the play pipe connection conical. The cylindrical surface of the tip should be kept highly polished. With the pump working at full capacity and an average bank, a jet 1¼ inches in diameter gives the best results.

"At a cost of about $15 per day a grading crew with this jet will move from 500 to 1800 cubic yards of earth in a day, the amount depending upon the character of the bank and the slope to which it is graded.

"It is believed that there should be added to each hydraulic grading boat a hydraulic dredge operated by a centrifugal pump to enable the grading of the bank to be carried on below the water's edge. One of the weakest points in the revetment system is the irregular position of the subaqueous mattress due to irregularity of the bottom.

"*Mattress Weaving and Anchoring* (Pl. 24 and Fig. 113).—The mattress, 70 to 90 feet wide and 12 inches thick, is woven continuously downstream on a barge fitted up for the purpose, lying normal to the bank, the inner edge of the mattress extending about 4 feet from the water's edge up the sloped bank.

"A specially designed boat, 25 by 70 feet, is used in mattress weaving. The lower gunwale of this boat is high, the upper one low, and raked to offer less resistance to the current. From the upper gunwale a calked platform, 13½ by 66 feet, rises on a slope of 1 on 3½, giving a working surface for the weaving. On this sloping platform are placed launching ways of 3 by 8 inch stuff, spaced from 6 to 10 feet apart. At either end of the platform outriggers are built for carrying the mattress beyond the width of the boat. At the rear end of the launching ways and on a level with their tops a brush platform, 12 by 66 feet, extends lengthwise the boat. Its lower edge is flush with the lower gunwale of the boat and it is supported 8 feet inboard by stanchions. The boat is stiffened longitudinally by a truss.

"On the sloping ways of this boat the mattress is woven. When the mattress has been woven to the top of the ways, the barge is pulled downstream from under the mattress until the edge of the completed mattress rests on the lower portion of the ways;

weaving is then resumed where it was left off and carried on until the top of the ways is reached again. A continuous mattress is thus secured.

"Straight, freshly cut willows, not less than 12 feet in length and from $\frac{3}{4}$ to $2\frac{1}{4}$ inches in diameter at the butt ends, are used in the mattress. One cord of willow brush will make about 140 square feet of mattress 1 foot thick. These willows grow on the sand-bars formed by the river and are the best material for the mattress, being very pliable.

"For starting the mattress a continuous bundle of willow brush 12 to 14 inches in diameter is made of a length equal to the width of the mattress and well wrapped with wire strand. Into this bundle the butts of the willows are forced at an angle of about 35 with the axis of the mattress. The mattress is woven very much as straw is plaited for various purposes, three to five willows being used in carrying the stitch. The ends of the willows projecting over the inner and outer edges of the mattress are turned in and well woven into the mattress, forming strong selvage edges. The brush for weaving is brought in wire bundles and laid crosswise, butts upstream, on the platform of the mattress boat. The bundles are then opened and the brush is passed one or more pieces at a time to the weavers.

"To give additional strength to the mattress, and for the purpose of anchoring it to the bank, a system of galvanized-wire strands, running lengthwise and crosswise of the mattress, is used. Both longitudinal and transverse members are composed of two $\frac{3}{8}$-inch strands each, made of No. 11 wires. One of these strands lies wholly underneath and the other wholly on top of the mattress. Both longitudinal and transverse wires are spaced 10 feet apart. All the longitudinal members are paid out under tension, as the mattress is made, from reels on the mattress boat, the top strand of each longitudinal passing through a fair-leader suspended some 18 feet above the brush platform.

"There are also transverse members of two $\frac{3}{8}$-inch strands each, extending one strand on top of the mattress and the other directly underneath it, from the outer selvage edge to deadmen on top of the bank back of the graded slope. They are laid out about normal to the axis of the mattress at a distance apart of 10 feet measured along this line. The transversals are run out from a reel on shore, the bottom part first, enough strand being pulled through past the outer selvage of the mattress to reach ashore when laid back on top of the mattress. Both parts of the transversals are laid just after the weavers pass the line, so that they are not woven into the body of the mattress. At all points of intersection of transversals with longitudinals the four parts of strand are brought together and fastened by stirrup bolts or clips of $\frac{7}{16}$-inch iron, but before the fastenings are made tight both parts of the transversals are put under tension from the outer-edge selvage edge to deadmen ashore by means of blocks and tackle. The deadmen are either rough blocks of stone, containing $2\frac{1}{2}$ to $3\frac{1}{2}$ cubic feet, or pile butts 12 inches or more in diameter and 4 feet long. The deadmen or anchors are planted 8 feet back of the top of the graded slope and from 3 to 5 feet deep, according to the character of

the ground. From the top of the slope to the deadmen in place the strands lie in a narrow trench dug for that purpose.

"The average day's work of a weaving crew is about 100 linear feet, costing from 60 to 70 cents per linear foot.

"*Sinking and Ballasting Mattress.*—Specifications for standard revetment provide that $\frac{3}{4}$ of a cubic yard of riprap stone per linear foot of mattress shall be used in sinking it to close contact with the bottom, the distribution being such that the weight per square foot of mattress increases from the shore out. It is also specified that an additional 50 cubic yards of stone shall be placed on 50 linear feet of mattress at the head and 1 cubic yard per linear foot on all laps. The stone is thrown from a barge, which is dropped downstream over the mattress with its outer end somewhat in advance of the shore end. It seldom happens that the full complement of stone is placed in sinking, as sudden shifting of the barge is often necessary to prevent buckling of the mattress, especially in a swift current. It is therefore usual to go over it a second time. The sinking is not carried closer than 150 to 200 feet above the mattress boat, and as much as 1000 linear feet, or even more, mattress is sometimes made before sinking is commenced. There is danger, however, in having so much mattress afloat, as a sudden rise may occur, and, though it might not otherwise damage the work, would possibly so foul the mattress with driftwood as to seriously impair its efficiency.

"The operation of spalling the inshore edge of mattress precedes the paving of the bank, and consists of filling well the interstices of the mattress from its inshore edge to the contour 3 feet below standard low water with small spalls or quarry chips and fairing up with the same material the shoulder formed by the edge of the mattress where it lies on the graded slope. This spalling serves to stop wave action through the mattress and to solidify it against ice in the event of loss of the paving or ballast. About $\frac{1}{16}$ of a cubic yard of spalls per linear foot is required.

"*Paving and Spalling Upper Bank.* (Pl. 24 and Fig. 114.)—The upper bank from the contour 2 feet above standard high water to standard low water, and as much lower as the existing stage of river permits, is covered with a paving of riprap stone 12 inches thick at standard low water, and 12 inches, 8 inches, and 6 inches thick, respectively, at the top of 1 on 2, 1 on $2\frac{1}{2}$, and 1 on 3 slopes. A covering 2 inches thick of spalls or fine quarry chips is put on the paving, thus completing the revetment. The paving is done from the top of the slope down, the stones being set up edgewise and placed with some care to make a compact covering, so that when completed ready for the spalls it presents an appearance of a reverse shingling. In this form it is well adapted to resist wave action and dislodgment by ice, driftwood, or other forces to which it is likely to be exposed. The spalls thrown on, at and near the top of the slope, are raked down over the pavement, filling the interstitial voids and finding lodgment in the paving surface.

"As will be seen from the above, the quantity of riprap stone for paving, as well as the spalls required per linear foot of bank, will vary with the height between the standard

high- and low-water planes, the height of bank when lower than 2 feet above standard high water, the stage at which the work is done, and to some extent with the grade of the slope, but the paving stone will average close to $\frac{1}{2}$ cubic yard per linear foot, and spall covering less than $\frac{1}{3}$ cubic yard for a height of 16 feet between high- and low-water planes."

Details as to the cost of this work will be found at the end of this chapter.

**Protection from High Wave-wash.**—Some valuable experiments in regard to protecting banks from high wave-wash were made on the Manchester Ship Canal between 1894 and 1898. The information, while not strictly pertaining to rivers, may afford some useful hints in cases where a city water front, a lateral canal, or similar construction has to be protected.

The ratio of the section of the canal to the immersed section of a vessel drawing $24\frac{1}{2}$ feet is about 4 to 1, and, although the speed is limited by the regulations to six miles per hour, small steamers sometimes attain speeds as high as 13 miles per hour. The material through which the canal was excavated varied from sandstone rock to soft, wet sand. Among the methods first tried was that of using heavy sandstone riprap, roughly trimmed so it would pack closely, and laid directly on the graded bank, on slopes ranging from 1 to 1, to 2 to 1, the latter being used where the banks showed a tendency to slide. Where this protection was carried up from the bottom of the canal it proved entirely satisfactory, and cost practically nothing for maintenance. It was, however, very expensive in its first cost. Another method tried was that of mattresses fastened to short piles and laid on a slope of $1\frac{1}{2}$ of base to 1 of height. This proved a failure, as the material washed out from between the twigs, and damage was also caused by decay. A third method was to use light sandstone riprap, but it required constant attention, as it was easily disturbed by the wave-wash. The method finally adopted was to drive short piles at the edge of the water, from 2 to 3 feet center to center, and joined by short walings. On each side of the walings was placed a toe of rough stone, just below the water, and supporting the protection. This consisted of a layer of limestone about 9 inches thick, laid on the earth, but closely packed by hand, the sharp angles of the stone assisting materially in holding it in place. This method has proved generally satisfactory, although in places it has been undermined by the wash from steamers. It was found that no injurious effect was produced upon the protection until the speed exceeded 8 miles per hour. A somewhat similar and equally satisfactory protection has been used on the Welland Canal in Canada; in this case the piles were spaced 8 or 10 feet apart.

The general conclusions drawn from experience on the Manchester Canal are as follows:*

(1) In excavation the slopes need not be protected under water for a depth greater than 5 feet, as that is the level of the plane of deepest erosion caused by wave-wash from passing craft.

* Paper by Mr. W. H. Hunter, 7th International Congress of Navigation, 1898.

(2) In embankments the entire slope should be protected.

(3) Above the water line the slopes should be protected against wave-wash for a height of 5 feet.

(4) The most effective protection consists of hard angular stone, of a size that one man can handle. Light-colored stone is better than dark, as it assists navigation at night.

(5) This protection should have a substantial toe 5 feet below the water; this toe may consist of a dike of stone, or of a simple berme.

(6) The stone from the toe to the water-level should have a slope equal to the angle of repose of the material; above that it may have as steep a slope as the material will permit.

(7) All bank protection should form a part of the original construction, as it is difficult to execute it satisfactorily afterwards.

On the Ghent-Terneuzen Ship Canal in Holland the experience was somewhat similar. This canal is cut entirely through a fine sand, with slopes of 3 of base to 1 of height; its depth is $21\frac{1}{3}$ feet, with a least ratio of water section to the section of the vessel of 3.9. The method finally adopted for protecting the banks was to use a covering of riprap for a few feet above and below water, placed on a slope of 3 of base to 2 of height and supported by a row of sheet piling, which was in turn supported by galvanized iron rods fastened to round piles about 10 feet apart, driven into the bank behind. Below the toe the slopes were made 3 of base to 1 of height and had no protection. Under the most severe conditions of soft soil and rapid passing of vessels no displacement of the slopes occurred below a depth of $6\frac{1}{2}$ feet.

On the Kaiser-Wilhelm Canal, connecting with the Baltic Sea, the general method adopted after much experimenting was to use below water a concrete composed of one part of cement to ten parts of sand (this was found to possess insufficient strength where exposed to the weather above water) and of a thickness of about $\frac{6}{10}$ of a foot, while above water a protection of roughly squared riprap or of brick was used. The brick gave quite satisfactory results. It was laid on edge on the graded slopes, with the courses parallel to the canal and in a single thickness; in many cases a half brick thickness was used and proved equally satisfactory. Wherever this protection was laid on a sandy bank it was found necessary to first place a bed of gravel or of spalls in order to prevent the wave action from attacking the slope through the crevices between the brick. This bed varied in thickness from 6 inches for a bank of pure sand, to 2 inches for a bank of sandy clay. Where many springs were found the concrete revetment was placed on a bed of gravel for the purposes of drainage, or riprap was used instead of concrete. The concrete, however, was cheaper than the stone, and cost from 72 to 91 cents per square yard. It was found desirable not to use steeper slopes than $1\frac{1}{2}$ of base to 1 of height.

**Protection by Vegetation.**—When a channel has to be preserved through a lake or an estuary certain species of reeds can sometimes be used to advantage in consolidating its

banks. One of the most important is known as the "Arundo phragmitea," which will grow well in fresh or brackish water and in a zone varying from 2 feet above mean water to several feet below.

The method of surface protection from ordinary wave-wash by means of sodding is described on p. 182, and also in Chapter VI under the clause "Protection."

FIG. 115.—Bank-head near Chamois, Mo., Nearing Completion. Looking Downstream.

**Bank-Heads and Groins** (Fig. 115 to 117, and Pls. 25 and 26).—On the Missouri River a special form of revetment was experimented with some years ago, in principle somewhat similar to the spur-revetment, but known as a "bank-head." It consisted of an isolated section of bank protection, of a size and distance apart depending on the local conditions. Those built were placed on the concave sides of bends, close to the edge of the bank.* Their use was based on the theory that by holding permanently

* Annual Report, Chief of Engineers, U. S. A., 1897.

points of the bank at certain distances apart the force of the current would not be able to cut in seriously between the protections, nor cut around them, owing to the shortness of the space in which its effects, such as scour, eddies, etc., had to work. On the

Fig. 116.—Bank-head at Rockport, Mo., as Reinforced. Looking Downstream.

Fig. 117.—Bank-head Near Chamois, Mo., Looking Downstream after Structure had Been Exposed to the Floods Two Seasons.

concave side of a sharp bend, for instance, if the points were too far apart, the velocity of the current would sweep against the bank between, and cut it away until it could attack and undermine the bank-head, but if the points were at a proper distance, the current

would work into the bank for a certain space, and the erosion would then cease, as there would not be room enough for further action. (See also pp. 179 and 189, etc.)

Experiments on the Missouri River showed that water of considerable amount and velocity would pass around a fixed curve of 300 feet radius without causing violent eddies, and that an angle of 30 degrees to the current was approximately the one at which a friable bank could approach a fixed or riprapped point without much erosion. With these data as a basis the bank-heads were built with a conical front, and a least radius of 300 feet. They were constructed of ordinary materials, as brush, stone, etc., and provided with an ample protection of riprap on the upper and lower sides. A large mass of riprap was also placed along the river-face, and renewed as the water undermined it, and it was found that this action ceased after a certain time, as the stone having fallen over gradually afforded protection to the bank below the limit of scour. (See Pls. 26 and 32.)

The first bank-head was built at Chamois Bend (Figs. 115 and 117 and Pl. 25), and they were later put in at several other points. An interesting study of their effects is shown on the drawing of Wilhoite Bend (Fig. *a*, Pl. 26), where a series of three, spaced about 3000 feet centers, was constructed in 1898. At that time the bank was closely tangent to each one as shown, but during the two following years the river continued its erosion and attacked their flanks, necessitating an extensive upstream protection of permeable pile groins and other reinforcement. These groins are shown by the fine lines close to the bank-heads. They appeared to remove the danger, and by 1900 the river seemed to have reached the expected limit of curvature, and the erosion to have ceased. That the attack was a severe one is shown by the recession of the bank line, and by the fact that along the outer face of the last bank-head the scour in the first season reached a depth of 35 feet below low water. It will be of interest to compare the positions of the apron after scour with those shown for similar cases in India. (Pl. 32.)

While this type of protection appeared to give good results in many ways in preventing bank recession, it was not proof against attacks from behind, and during unusual rises some of them were flanked and destroyed. The failure of appropriations put an end to the extension of the works before sufficient time had elapsed to afford conclusive results. Although this series was largely experimental, it was stated that the essential parts of the works as finally built could be reproduced at a cost which would not exceed one-half that of a continuous revetment. Whether or not the sharp curvature of channel produced by these bank-heads would be prejudicial to navigation is a matter about which no data appear available.

A method of bank protection, or rather of river training, based on principles similar to those of the bank-heads, has been employed in India by Mr. P. Denehy, where the structures are known as Denehy Groins. Each one consists of a stem-dike of unprotected earth (the top of which must of course be above high-water level) like a levee, running

out towards the river from the edge of the "khadir" or bottom lands, and provided with a tee-shaped head covered with stone and guarded by an apron of stone which falls in and protects the work as the river undermines it, just as with the bank-heads. The river encroaches on the spaces between the groins, but must never be allowed to cut in far enough to flank the stone-protected end. To prevent this, the upstream arm is made three or four times as long as the downstream one.

A good example of their work is seen on the drawing of the system at the Narora Headworks of the lower Ganges Canal, where a series was put in to prevent flank attacks upon the weir (Pl. 27). The groins are about half a mile apart along the river, and 3000 feet apart measured across it, with heads 300 feet long. They extend for four miles above the weir on each bank, and for 16 miles below on the right bank only, where they protect the main line of the canal. In all there are 46 groins, composed of 45 miles of earthen embankments, besides the tees. The fall of the river there is 12 to 14 inches per mile, and the bed composed of very fine sand, indicating possibilities of severe scour. By adding stone where needed, however, the works have been gradually brought to stability, and are said to fulfill their object with complete success.

One feature of the system to be noted is that the first or upstream groin must be thoroughly protected against flanking in the same way as the bridge embankments described further on in this chapter, since without this precaution there would be nothing to prevent the river from breaching the one furthest upstream. At Narora this protection was afforded by the bridge, as shown on Pl. 27, while its absence at a similar system of works on the Indus, at the city of Dera Ghazi Khan, caused disaster. The river at that point, as shown on Fig. 1, Pl. 28, was gradually encroaching towards the city, and after several expedients had been tried without avail to hold it in check, commencing about 1876, a system of Denehy Groins was built in 1897. These were of the usual type and composed of embankments of material taken from the river-bed, and running out from the main river bank to stone-protected tee-heads. The river, however, soon commenced to attack the heads as it worked steadily westward, and considerable additions of stone and lengthening of the tees had to be carried out. However, on the 11th of July, 1899, the main current reached the outer end of No. 4 Groin, and was fast swirling in to attack it from the rear. Stone and brush were rapidly put in, but gave only temporary relief, and on the 18th the eddies of the river broke through the stem embankment, and in a short time swept down the remainder of the head and most of the embankment itself. Groins 1, 2, and 3 were thus left defenseless, and in the floods of 1900 they were destroyed in a manner similar to No. 4, and the city was left to its fate. Fortunately the river suspended its westward flow shortly afterwards and commenced a periodic swing toward the other shore, thus relieving the immediate danger,* but a few years later it commenced to return. Owing to the great cost of providing adequate

* "River Training and Control," Francis J. E. Spring. The accompanying information and the plates are reproduced by courtesy of Mr. Spring.

protection works it was considered preferable to permit the inhabitants to move, which they accordingly did, leaving the town to its fate.

A method of securing the upper end of such a system was suggested as shown on Fig. 2, Pl. 28. The action of the circular head in protecting the groins would be similar to that described further on for the "Bellbunds."

**Bank Protection on the Volga.***—On this river, which resembles the Mississippi in size, but is of more tranquil regimen, bank protection has been widely applied, as the banks in many places consist of fine sand and are easily eroded. The chief method used consists in grading the slope, the hollows being first filled with brush, and driving stakes about $3\frac{1}{2}$ feet apart, their tops projecting 2 feet above ground. Brush is then wattled about them so as to make a kind of hurdle fence, and the spaces are filled with riprap level with the tops of the stakes. Where the ground is of very untrustworthy material, a thick layer of straw is first placed upon it. Fascine mattresses 2 feet thick are used to extend the protection below the water-line, but their width rarely exceeds 50 feet.

**Protection on Indian Rivers.** †—On the rivers of Northern India, such as the Indus, the Ganges, the Brahmaputra and their tributaries, a brief description of whose characteristics was given at the end of Chapter I, a system of training has been evolved in connection with the railway bridges and irrigation weirs which is of great interest. (See also p. 228.) Although the methods will rarely be found applicable to rivers in Europe or America, a study of them will afford many useful lessons. The rivers in question, while of considerable size and extreme mobility of bed, restrain their wanderings within limits usually only a few miles across, although the "khadirs" or bottom lands extend far outside of these.

The problem before the Indian bridge engineers was how to determine the limit to which the river could be contracted so as to shorten as much as possible the length of the bridge. This involved also the questions of the protection of the foundations from undermining and of the earth approaches from erosion. The problem before the irrigation engineers was how to protect their weirs from being flanked. The earthwork approaches to a bridge, built out from the limits of the high land, although often of enormous yardage were comparatively inexpensive, while the bridge piers and superstructure were costly, and had therefore to be reduced to the least opening which would safely pass the floods, the last desideration including a training of the river which would keep it during its wanderings from attacking the unprotected slopes. The manner in which this was liable to happen, and actually happened at certain of the earlier works, is illustrated by Figs. 1 to 11, Pl. 29. The earthen approaches, composed chiefly or entirely of sand from the river-bed, caved rapidly under any attack of the current, and the expense of protecting them with stone—the only method which

* Report on the Volga, Major F. A. Mahan, U. S. A., 1904. See also p. 84 and after.

† "River Training and Control," Francis J. E. Spring.

has been found successful in the climate of India—would have been so great that means were sought for preventing the river from reaching them. As happened in many instances in America, the early efforts were hampered by lack of funds, and often by lack of experience, and it was only after some twenty or thirty years of experiments of many kinds that a system was evolved which appeared to solve the problem satisfactorily. Figs. 5, 6 and 7, Pl. 29, show the first methods tried, consisting of unprotected levees or "bunds" with stone-protected heads which the river could not flank, but the upstream one of which remained vulnerable until the series had been extended to intercept the edge of the bottom which confined the river's wanderings. Thus at the bridge over the Chenab, near Wazirabad, built between 1870 and 1875, the spurs by 1902 had been extended to cover a length of $1\frac{1}{4}$ miles on one bank and $3\frac{3}{4}$ on the other, and necessitated a considerable sum annually for their upkeep. Such extensions, however, were very expensive, and the river might at any time find a weak spot and destroy a portion of the works. An example of this is shown in Figs. 1 and 2, Pl. 30, where the river swung in and breached both rows of spurs, destroying 4000 feet of the right embankment and a considerable portion of the left one. (See also Fig. 1, Pl. 28.) The system was then remodeled as shown by Figs. 3 and 4, Pl. 30, the latter one showing on the left bank an actual example of an attack by the river on the "Bell bunds." These guide banks (called "Bell bunds" after their projector, Mr. J. R. Bell, and shown in general plan by Fig. 8, Pl. 29) represent the evolution of the various systems of training, and were first used about 1888 at the Sher Shah bridge over the Chenab. Their effect is illustrated by Figs. 9 to 11, Pl. 29; they act as guide banks or gorge walls which the river cannot undermine because of the riprap armor, and which it cannot outflank because of the riprapped heads. They thus form permanent high banks through which the river must flow, however much it may wander from side to side in its approach. The principle of their action lies in the fact that the river changes its course by commencing a curve which lengthens as the erosion continues, the upstream and downstream limits gradually approaching each other in a loop, until only a narrow neck of land is left between. If the erosion is allowed to continue, the intervening neck of land is finally cut through, forming a cut-off, and the curved channel is abandoned and all erosion in it ceases. If therefore the erosion in this curved channel can be kept at a sufficient distance from the works to be protected, a cut-off will eventually occur and Nature herself will remove the danger. Thus in Fig. 9, Pl. 29, is shown a curve ready for this change; it is limited on the one hand by the permanent river bank, and on the other by the head of the guide bank, which it is endeavoring to outflank. These two limits will prevent the channel from reaching the embankment, since the river has reached the sharpest curvature in which it can flow, and in another flood or two a cut-off will occur at *L*, and the river will swing over to another course, as shown in Figs. 10 and 11.

The construction of these "bunds" or dikes is of course carried out during low water, and has to be completed if possible in one season, and certainly before much change has

taken place in the channel of approach. Where this is not practicable, the wandering of the river may necessitate a considerable change of plans to suit the new location, and this actually occurred in certain cases. The guide banks are built on the dry portion of the bed and of sand taken from the spot, and experience advocates a crown width of 20 to 30 feet, placed 4 to 5 feet above extreme flood level, and with slopes of 2 horizontal to 1 vertical or flatter. The river faces and the upstream and downstream ends are carefully covered with hand-placed riprap, and in front of them a wide level trench is excavated as near low-water level as possible, in which an apron of riprap is laid. It has been found very advantageous, as in the United States and elsewhere, to place the stone on the slopes on a layer of spalls or brickyard refuse, as this prevents the current from washing out between the pieces the sand of which the bank is composed. When the river in course of time impinges against the guide bank at various points, the stone in the apron falls down as the erosion progresses and protects the works, and eventually there will be found a continuous layer of riprap from bottom to top. The general design is illustrated by Fig. 2, Pl. 31. The rear slope is sometimes covered with earth, as are also the slopes of the railway embankments, so as to encourage the growth of a vegetation which will hold the surfaces against erosion by rain or waves. It has also been found a useful practice to place small culverts 8 or 10 square feet in area in these embankments; these have the effect of creating a slow current behind the guide banks during floods, causing a steady deposit of silt there and a gradual upbuilding of the former bed.

The stone used is "one-man riprap," the pieces weighing from 60 to 120 pounds, the latter being about the limit of weight which the native laborer can handle. Such stone has been found to stand successfully currents as high as 18 feet per second. As before mentioned, this is the only material suitable to these rivers; the brush and mattresses used in America would be prohibitive in price, besides being unfitted to cope with these vigorous and ever-changing streams. The stone costs usually from 50 cents to $2 per cubic yard delivered. In the earlier days tree-trunks, stone in nets, and many other expedients were tried, and circular wells of brick sunk like bridge piers were proposed, but the simple protection of riprap properly designed and laid has been finally adopted as the best and most reliable method.

The crossing of the river, as before stated, comprises the problems of finding the shortest opening needed to pass the floods, and of preventing the river from outflanking and undermining the bridge. In the case of protection for a weir, only the problem of outflanking has to be dealt with. In its natural state the river wanders within the limits of bottom lands from one to many miles across, which it covers with water during floods, all of which must be made to flow under the bridge. With streams whose beds consist entirely of sand, often of the lightest quality, and whose flood discharges mount up into hundreds of thousands of cubic feet per second, this problem is a serious one. Its successful solution depends upon the erosion of the bed; the first flood occurring after the river has been confined scours out

the bed between the guide walls and under the bridge, and thus restores the natural area very quickly, and changes it at the same time to one with a better hydraulic radius. The afflux, or swell-head, due to the confinement, is but temporary; the swifter the current caused by it, the swifter is the erosion, and a single season usually suffices to regain the needed area. The erosion is of course not uniform; local influences have their effects, and each flood may cause some change of depth. Each portion of the works will probably thus be exposed to deep scour, and must therefore be contructed so it will stand unharmed. A practical example of this narrowing of a river was that of the Bezwada bridge over the Kistna; the natural flood cross-section of the stream was 136,000 square feet, and the works reduced this to 118,000 square feet. During the first flood the latter area was found to have increased to 126,000 square feet, but as observations failed to detect any afflux it was apparent that owing to the improved hydraulic radius of the bed, produced by the concentration, the river was passing the same amount of water which formerly had required the larger but more irregular area. The scour in this flood was equivalent to an average increase in depth of $4\frac{1}{2}$ feet. At other bridges this average increase during the first flood has been found to be 10 or 11 feet, with local depths scoured to 25 feet. While the scour is naturally greatest under the bridge and between the training works, succeeding floods slope the basin for a mile or two up- and downstream, thus providing for themselves an easy approach and outflow.

To find the limit to which the river may be narrowed Mr. Spring gives the following rules, deduced from actual experience:

"1. Make at least three fairly representative cross-sections of the entire width of river, showing high and low water.

"2. Ascertain the fair average existing bed slope applicable to each cross-section.

"3. Prepare diagrams of Kütter's or the Mississippi formula suitable for the ascertained slopes.

"4. Applying the diagrams, work out the total flood discharge for the separate sections, and either by averaging or by judgment come to some conclusion as to what is the discharge of the river. It does not really matter very much, for the purpose in view, if the discharge so calculated should not be very correct.

"5. On the section where the river is to be narrowed draw a revised ideal section, deeper all over than the existing one by several feet, according to the quality of sand.

"6. Pick out the best part of this deepened section and, applying the velocity diagrams referred to at (3), find out how much of it will suffice to carry the estimated discharge. The part so arrived at will suffice—and indeed more than suffice—to carry the entire river, and the remainder of it may safely be blocked up.

"7. Should further refinement be wanted, it may quite safely be assumed, as indeed is borne out by experience, that instead of the river deepening equally all over, its deepest part will probably become 20 or 30 feet deeper instead of the assumed depths, while its shallower parts will probably not deepen more than a foot or two. Perhaps, in an

exceptional year, one-third of the section may deepen a good deal and the rest very little. Draw a new fancy section accordingly. See Fig. 1, Pl. 32.

"8. Then apply the velocity diagram to the new section and see whether the area got by (6) may not be still further reduced.

"9. The greatest depth of scour arrived at by (7) may be taken, more or less, as the clue to the depth to which the piers must be sunk. For this depth of scour may occur anywhere in the length of the bridge in successive years, and each and all of the piers had better have as much of their length in the sandy bottom as will suffice to hold them up against the current due to the maximum depth of water to which they are liable to be exposed.

"10. The depth arrived at in (7) is also to some extent the clue to the quantity of stone that must be used as armor for the sides of the artificial gorge designed to lead the river between the abutments of the bridge."

A diagram showing the comparative areas thus obtainable is shown on Fig. 1, Pl. 32.

To ascertain the greatest depth of scour, which in this class of river may reach from 50 feet below low water for those whose beds are of coarse sand, to 100 feet for beds of fine silt, soundings are taken at all likely points within say 10 miles of the site, and the deepest hole found may be considered as the probable limit of scour. These soundings should be taken in the bends where normal conditions exist, as abnormal conditions or obstructions of any kind may give rise to swirl scour such as properly designed guide banks will not create, but which may bore down to depths 50 per cent greater than those which would be found under circumstances of ordinary flow. There is of course an intimate relation between the slope of the river and the quality of the material of its bed, and from these two elements the depth of probable scour might be determined, were sufficient data available. As observations are usually lacking, however, the method of sounding has been employed instead and has proved sufficiently reliable.

The actual location of the guide banks is determined largely by the location of the river at the time of construction, as the banks and aprons must be laid on dry land. Experiments with models show that a more uniform scour is produced by having the banks converging upstream; but it is rarely practicable to do this owing to local conditions.

The shape of the heads of the banks is of the utmost importance, since they must be so designed that the river cannot undermine them, and they must cause no eddies in the currents. They must be placed at such a distance above the bridge, based on the radius of the sharpest bend in the neighboring sections of the river and on the limit of the wanderings of the channel, that the current cannot possibly wander within say 200 or 300 yards of the embankment. The upstream length of bank as thus determined may be from three-quarters to the full length of the opening of the bridge, and should extend about an eighth of a mile downstream. There is practically no danger from erosion

below the bridge. For the radius of curve of the head of the bank Mr. Spring gives the following table, applicable under general conditions:

| Sand Classification. | Probable Maximum Abnormal Scour. | Fall per Mile of River, in Inches. | | | | |
|---|---|---|---|---|---|---|
| | | 3 | 6 | 9 | 12 | 18 |
| | | Radius of Upstream Curved End of Guide Banks. Feet. | | | | |
| V.C. | Under 20 feet | 200 | 250 | 300 | 350 | 400 |
| V.C. | Over 20 feet | 250 | 310 | 375 | 440 | 500 |
| C. | Under 30 feet | 300 | 360 | 425 | 490 | 550 |
| C. | Over 30 feet | 350 | 430 | 510 | 590 | 670 |
| M. | Under 40 feet | 400 | 425 | 550 | 625 | 700 |
| M. | Over 40 feet | 450 | 550 | 650 | 750 | 850 |
| F. | Under 50 feet | 500 | 590 | 675 | 760 | 850 |
| F. | Over 50 feet | 600 | 725 | 825 | 925 | 1020 |
| V.F. | Under 60 feet | 600 | 700 | 800 | 900 | 1000 |
| V.F. | Over 60 feet | 800 | 900 | 1000 | 1100 | 1200 |

Radius of curve of downstream ends of guide bank to be half the above, with for minimum radius, that on which maintenance trains can be run.

The figures in the first column refer to the following grades of sand: "C., or coarse, is when 80 per cent of the whole is stopped by the 40-wire and coarser sieves; V. C., or very coarse, is *coarse* sand when 25 per cent of the whole has been stopped by the 16-wire and coarser sieves; F., or fine, is when 80 per cent of the whole passes through the 75-wire sieve; V. F., or very fine, is *fine* sand when 20 per cent of the whole passes the 100-wire sieve; M., or medium, is grades between coarse and fine."

The wires refer to the number of wires or meshes to the inch, a 40-wire sieve having 40 meshes to the inch, or 1600 to the square inch.

For the thickness of the riprap or "stone pitching," the following table is given:

| Fall per Mile of River in Inches. | 3 | 9 | 12 | 18 | 24 |
|---|---|---|---|---|---|
| Sand Classification. | Thickness of Stone Pitching, Inches. | | | | |
| Very coarse, V.C. | 16 | 19 | 22 | 25 | 28 |
| Coarse, C. | 22 | 25 | 28 | 31 | 34 |
| Medium, M. | 28 | 31 | 34 | 37 | 40 |
| Fine, F. | 34 | 37 | 40 | 43 | 46 |
| Very fine, V.F. | 40 | 43 | 46 | 49 | 52 |

The general shape of the apron, based on the foregoing data and on results of actual experience, is shown by Fig. 4, Pl. 32.

The steepness to which the current of these rivers occasionally scours the banks, considering the material, is remarkable. Instances are on record where for long distances the sand has been cut to cliffs practically vertical both above and below the water level, and measuring 60 feet from base to summit. An apron of light-weight stone on the edge of a steep sand-cliff under water appears to have little effect in breaking it; but one of heavy stone seems to force down the edge and hence falls over more gradually and

produces a more reliable protection. The actual position taken by an apron is shown by Pl. 32. (See also Pl. 26.)

The bridge piers to be exposed to the scour are formed of circular cylinders or wells of brick, sunk like open caissons by excavation from the inside, carried out by grapple buckets. Where the site lies in water, an island of riprap is usually first made and the interior filled with sand to above water level, and on this surface the brickwork is begun. The former practice of using several wells 10 or 12 feet in diameter to compose one pier, and sunk to a depth of 50 to 75 feet below low water, has been replaced by the use of wells 20 feet or more across, with walls 6 or 7 feet thick, and sunk to 100 feet or more as needed. The shallow piers of the early bridges have had to be protected against scour by beds of riprap, much of which in certain cases has had to be replaced after each flood. That the danger of being undermined was serious was shown by the loss during construction of several of the wells of the Alexandra bridge over the Chenab, which were overturned and lost, although sunk to 75 feet below low water. For the depth below low water, of piers in sandy beds Mr. Spring gives the formula: " Depth should equal maximum ordinary scour plus rise of flood." Riprap placed as a circular apron 2 to 3 feet deep is also recommended to be used around the piers in amounts sufficient to counteract the effect of the swirl scour which they may create.

In some cases the widths of earlier bridges have been reduced with advantage, as was done with the bridge just referred to, which was narrowed between 1888 and 1890 from 64 spans 142 feet centers to 28 spans of the same length. The lengths of certain smaller bridges, proposed in 1884, but not built until about 1900, were reduced as the result of intervening experience from one-half to two-thirds of the lengths originally proposed.

Plate 31 shows the preliminary design for the training works of a bridge over the Ganges near Allahabad, now known as the Curzon bridge. The various structures were completed in 1908. The spans were made 15 in number and of 200 feet opening, giving a clear width of 3000 feet, the main river being about $1\frac{1}{4}$ miles wide. A single guide bank was used 4000 feet long (of which 2300 feet were upstream of the bridge), connected to the shore with an arm at about right angles. This arm was made 3000 feet long, and joined the main guide bank in a curve of nearly 600 feet radius. The river slopes of the guide bank were made 2 horizontal to 1 vertical and were covered with riprap 4 feet thick. The crown was made 20 feet wide and 5 feet about extreme flood, and the land slopes were planted with grass. The earthwork of the bank contained nearly two million cubic yards, and at times required the labor of 7000 men. The total cost of bridges of this type or of the weirs together with the training works may run from a quarter of a million to five million dollars.

The cost of guide banks of certain completed structures is given as from $25 to $60 per lineal foot of bank.

**Cut-offs.**—The prevention of cut-offs, the causes and effects of which have been described in Chapter I (pp. 27, 38 and 42), is a matter of great importance on rivers

where property or structures have to be protected, or stability of channel maintained. The local shortening of the river's course and the consequent sudden increase of fall leads to swifter currents and accompanying erosion, to the formation of new bars and a derangement of channels, and to a general and sometimes violent readjustment of conditions over long distances. These changes may lead to erosion at vital points, as was the case in the lower Mississippi at Memphis, where a cut-off occurred about 20 miles above the city in 1876. The increase in slope led to rapid alterations along the neighboring portions of the stream, and the bank at Hopefield Bend, close to the upper end of Memphis, receded between 1100 and 1800 feet in six years, while opposite to it, on the city side, the bank filled out with equal rapidity, creating an enormous sand-bar. Thus the upper end of the water front was threatened with sanding-up, while the altered currents just below began to undermine the railway and house property. The velocity of the stream at times was 8 miles an hour. The banks by degrees were successfully protected by revetments, but annual extensions have been necessary in order to guard against further encroachment. (See also p. 24.)

On the Indus and certain other rivers of northern India where for long distances the channel is divided into a multitude of side streams and outlets through which the river begins to run even in moderate stages, cut-offs (or "avulsions" as the Anglo-Indian engineers term them) are opened very readily. The natives, however, where their land is liable to be washed away or injured by such changes, build dikes or levees across the side streams and by this means assist in holding the river in its main channel, and gain in addition valuable deposits of silt.* The influence of their primitive training works in assisting the river to obtain and keep a stable channel is said to be considerable. To quote a contrast to this practice there have been instances in America where the farmers have cut ditches across the narrow necks of land and thereby artificially produced cut-offs in the belief that they would permit dangerous floods to pass off more rapidly. There was at the same time much complaint among them because of the great erosion of the banks during floods, and a demand that more cut-offs should be made in order to let the water pass off still more quickly.

Cut-offs are immediately due to one of two causes, viz., longitudinal erosion of the banks—the caving going on from each side until the neck is pierced—or surface erosion, which usually begins at the lower side and works back as shown by Fig. 118. This cut represents the effect of a single flood at Leland's Neck on the lower Mississippi, where the washout reached dangerous proportions, and a cut-off was prevented only by the presence of a stratum of hard clay about 10 feet below the surface and by the recession of the flood. Sometimes both causes are at work together, but where the erosion along the banks has been neglected until a cut-off is imminent a complete revetment as well as a spur-dike may be required for safety. In many cases where the neck is still wide the immediate danger arises from surface scour; this can be averted by building a spur-dike

* Physics of the Indus, R. A. Molloy. See also Chapter I, p. 39 and after.

(whose crown must of course be above flood level) along the neck to prevent the water from flowing across. The outlying end has usually to be specially protected with plenty of one-man riprap, as the water will pour round it very swiftly. For ultimate safety, however, the caving banks and the surface will each require attention. Fig. 119 shows a case where revetment and dike were both employed close to the city of Natchez on the lower Mississippi.

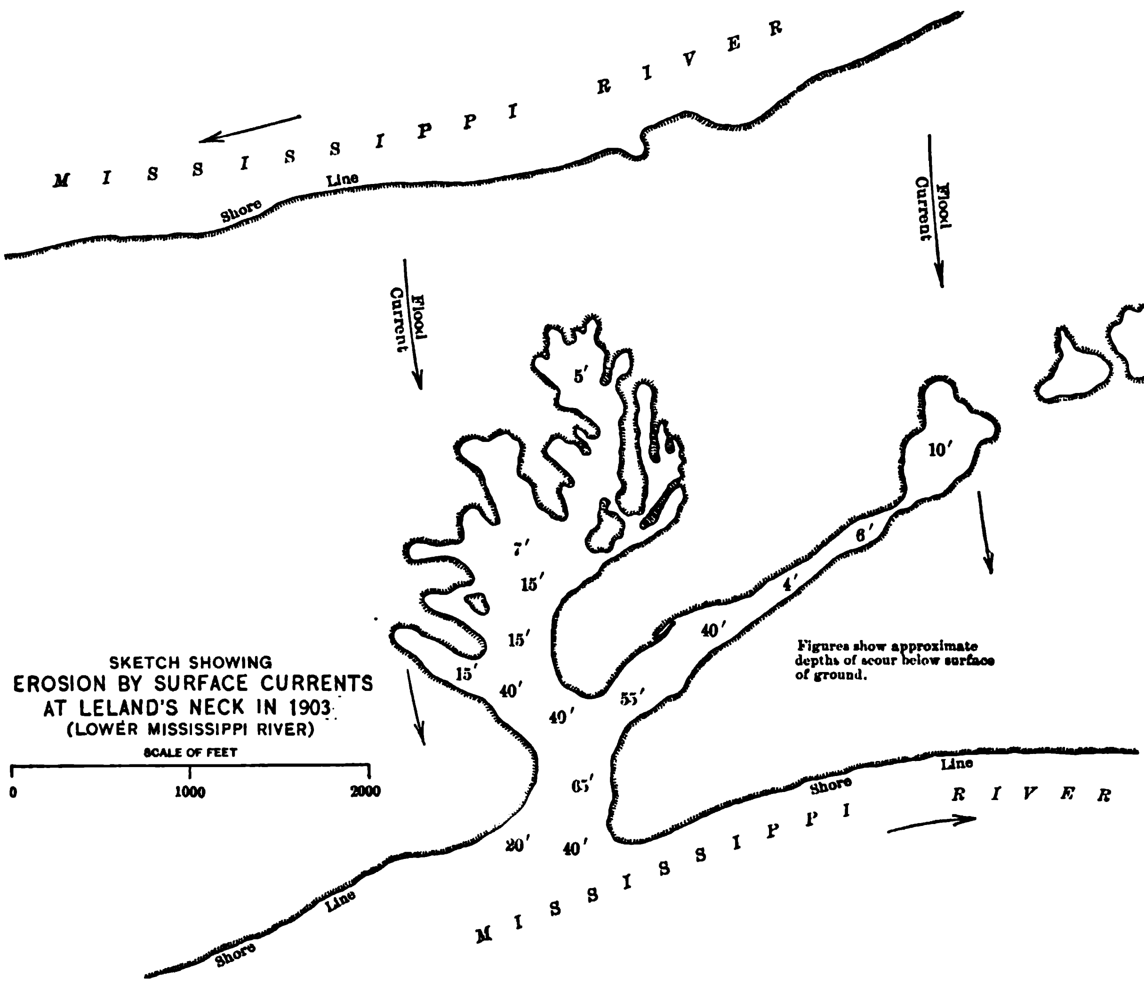



FIG. 118.

The direction of the currents in a flood tending to produce cut-offs is shown by Fig. 120, which represents a portion of the lower Mississippi. From gauge readings taken at the ends of this section at Arkansas City and Greenville, it appeared that in high floods the slope along the main channel, following the windings of the river, averaged 0.31 foot per mile, while the slope along the levees, following almost a direct line, was more than twice as great.* These conditions produce very swift currents across the necks, and in 1903 only the most strenuous work prevented several cut-offs from occurring

* Journal, Memphis Eng. Soc., vol. iv, No. 1. A. Miller Todd.

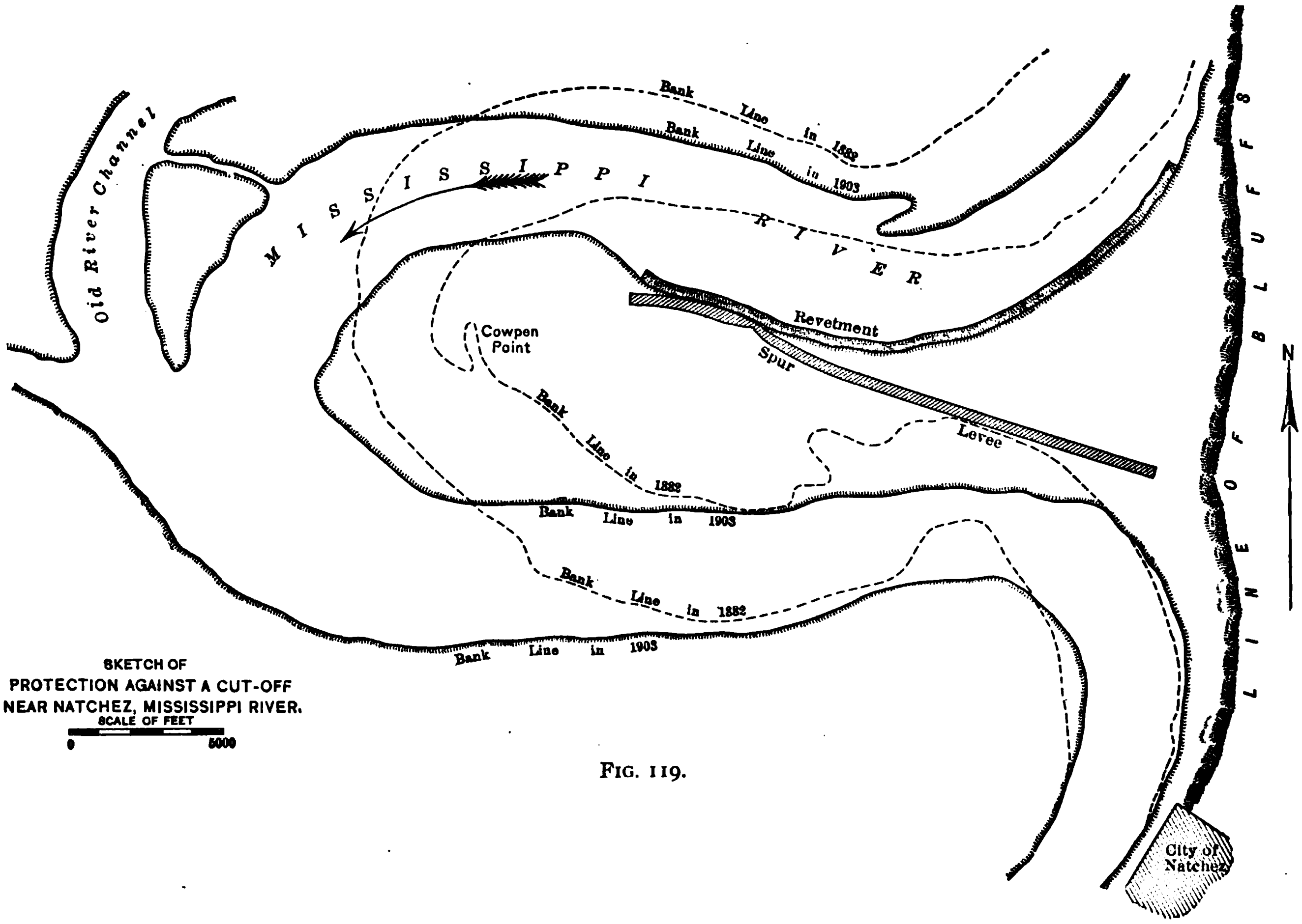



FIG. 119.

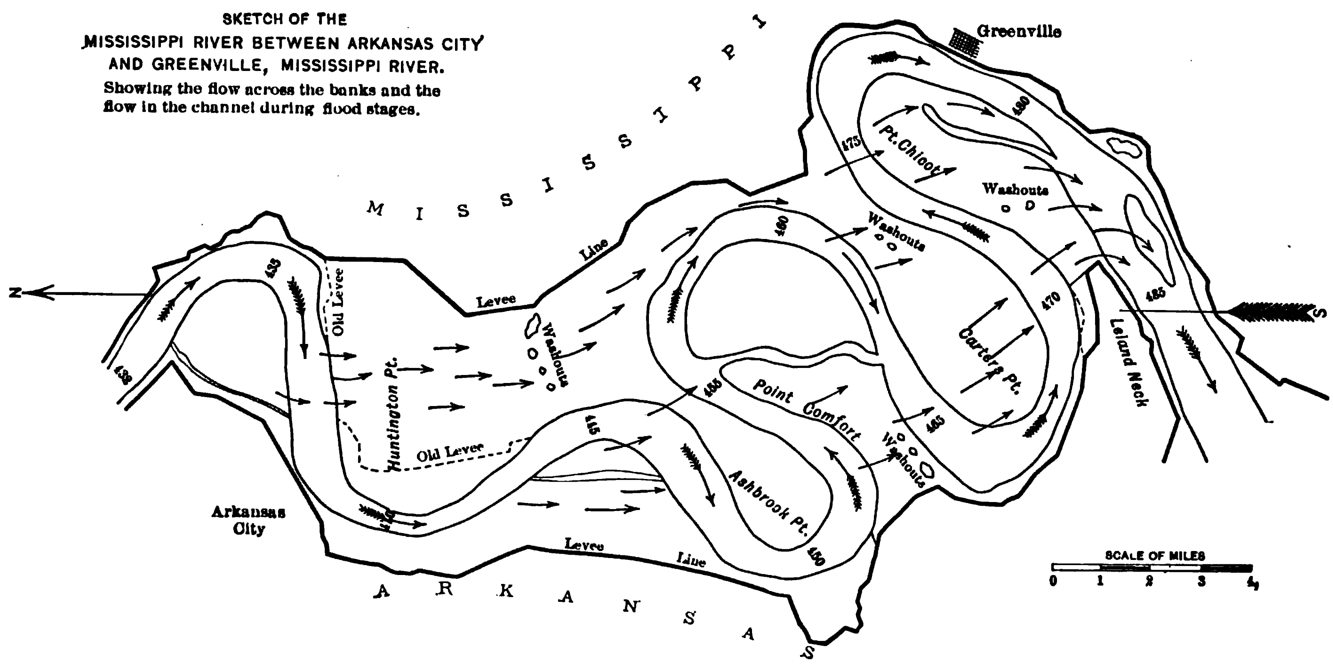



FIG. 120.

by surface scour. There were at that time no spur levees to prevent the flow, and the washout holes shown by the figure indicate the commencement of the erosion. The presence of trees and brush on a neck has been found of considerable value in preventing a cut-off, as they retard the swiftness of the flow.

**Sills.**—Subaqueous sills are at times employed to prevent a river from enlarging its channel. They extend from bank to bank, each end being well protected to secure the work against flanking. Several examples are to be found on the Mississippi, the earliest ones being those constructed during the improvement of the South Pass by jetties, between 1875 and 1879. They were placed at the "Head of the Passes," where the main stream of the Mississippi divides into the three large branches of its delta, and had as their object the prevention of the enlargement of the Southwest Pass and of Pass à l'Outre during the time in which the discharge through the mouth of the South Pass

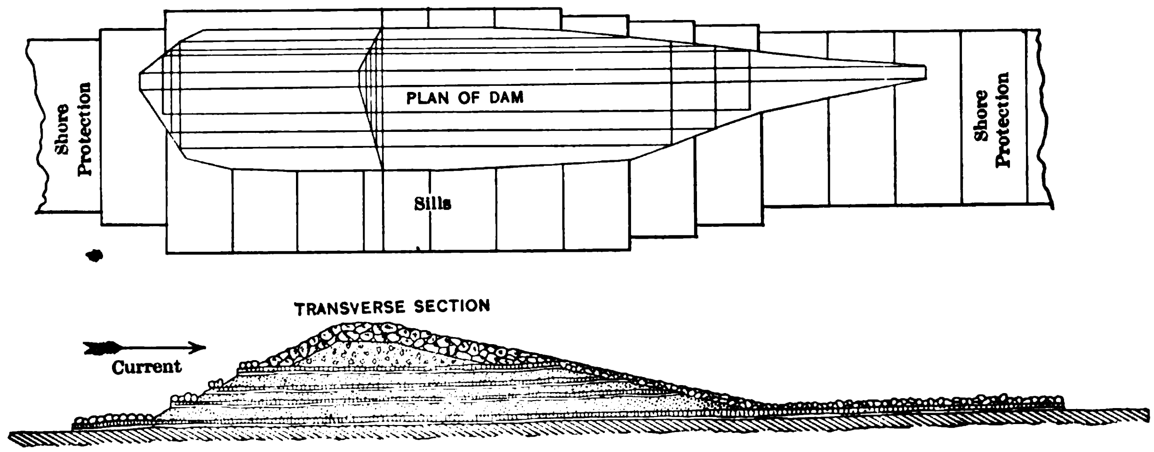



FIG. 120*a*.

would be obstructed by the contraction of the jetties. These sills were of the ordinary mattress type ballasted with stone, and were from 50 to 75 feet wide. Between 1900 and 1907 new mattress sills were completed across Pass à l'Outre, the largest one having a width of 200 feet and a length of about 4000 feet (Pl. 33.)

A similar protection of mattresses was used in 1887 near the head of the Atchafalaya River (one of the outlets of the Mississippi), where there appeared evidences of a tendency to enlarge the channel. This would have been followed by a greater diversion of flow and in all probability by ultimate disastrous changes. Two willow mattress sills were used, each 3 feet thick and 300 feet wide, heavily ballasted with stone, extending up to the high-water banks, and placed about 1800 feet apart. These sills have proved very successful in preventing any enlargement of the river-bed, and were reported in 1910 to be in satisfactory condition.

Two submerged dams of somewhat similar design were built on the same river near Simmesport, Louisiana, in 1890 and were in good condition twenty years later. The cross-section and plan of one is shown by Fig. 120*a*. The dams are built of alternate

layers of willow mattresses and gravel mixed with clay, with riprap coverings, the natural foundation being of fine sand. The centers are about 5 feet below low-water level, the river being about 50 feet in depth at that stage. In floods the water rises 50 feet above the crests and pours over them with a velocity of 20 feet a second. The discharge at such stages is estimated to be 500,000 second-feet.

**Cost of Bank Protection.**—The cost in 1905 of the small fascine mattresses used for subaqueous protection on the upper Mississippi was from 40 to 60 cents per cubic yard in place. The average amount of fascines used in a total of 188 miles of shore protection, built between 1878 and 1904, was 1.55 cubic yards per foot run, and of riprap, 1.73 cubic yards per foot run. The total amount of material used in the work was 3,275,000 cubic yards, or 3.28 cubic yards per foot run.*

The cost of 10,000 square feet of subaqueous willow and lumber mattresses used on the Mississippi near St. Louis between 1896 and 1900 as given in detail below, averaged 4.8 cents per square foot for the willow mattresses sunk and ballasted, and 2.6 cents per square foot for the lumber mattresses.

COMPARATIVE TOTAL COSTS IN PLACE OF BRUSH AND LUMBER MATTRESSES, PER 10,000 SQUARE FEET. MISSISSIPPI RIVER, BETWEEN ST. LOUIS AND CAIRO, 1896 TO 1900.

| Material | | Brush. | | | Lumber. | | |
|---|---|---|---|---|---|---|---|
| | | Quantities. | Unit Cost. | Total. | Quantities. | Unit Cost. | Total. |
| Brush | Cords | 83 | $1.60 | $132.75 | | | |
| Lumber | Feet B.M. | ........ | ........ | ........ | 8277 | $0.016 | $94.98 |
| Stone | Cubic yards | 106 | .95 | 100.31 | 38 | .93 | 35.32 |
| Rope | Pounds | 66 | .09 | 5.91 | 8 | .09 | .73 |
| Wire | Pounds | 347 | .23 | 7.81 | 373 | .03 | 11.39 |
| Nails and spikes | Pounds | 131 | .02 | 2.62 | 123 | .02 | 2.65 |
| Miscellaneous | Pounds | ........ | ........ | 2.13 | ........ | ........ | 1.60 |
| **Labor** | | | | | | | |
| Labor and superintendence | | | | 104.94 | ........ | ........ | 46.30 |
| Subsistence (exclusive of service) | | | | 41.21 | ........ | ........ | 16.99 |
| Quarter boats and mess outfits | | | | 9.12 | ........ | ........ | 4.32 |
| **General Expenses** | | | | | | | |
| Office, and general expenses | | | | 13.46 | ........ | ........ | 10.20 |
| Depreciation and care of plant | | | | 41.22 | ........ | ........ | 27.96 |
| Steamboat service | | | | 8.77 | ........ | ........ | 4.19 |
| Care of materials | | | | 9.47 | ........ | ........ | 3.32 |
| Total cost per 10,000 square feet | | | | $479.72 | ........ | ........ | $259.95 |
| Cost per lineal foot of mattress, 125 feet wide | | | | 6.00 | ........ | ........ | 3.25 |
| Cost per square foot | | | | 0.048 | ........ | ........ | 0.026 |

On the lower Mississippi the cost of fascine mattresses sunk and ballasted runs from 5.5 to 7.0 cents per square foot, an average being 6.0 cents, equivalent to $18 per linear foot of bank for mattresses 300 feet wide. The cost of obtaining brush and making about three million square feet of mattress in 1903 averaged 3.7 cents per square foot, and the cost of sinking, including the expense for ballast, from 2.4 to 3.1 cents per

* Annual Report, Chief of Engineers,. U. S. A.

square foot. The amount of ballast used to each 100 square feet was about 0.72 ton. The grading of about 4200 lineal feet of bank by hydraulic process in connection with this work was from \$1.39 to \$1.78 per lineal foot, and the cost of paving the banks with concrete was from 4.9 to 6.5 cents per square foot for a total of 187,600 square feet. The total cost per lineal foot of bank for an average case, including mattresses, grading, and paving, where the paving, etc., was of full extent, was about \$25 net, or, including care of plant, repairs, administration, etc., about \$31. This is equivalent to about \$155,000 per mile. It is stated that the cost of standard revetment averages about \$18 per linear foot for the subaqueous work (300-ft. mattress) and \$11 for the paving, making a total of \$29. The average itemized cost per 100 square feet in 1905 was as follows:

| | |
|---|---|
| 1.40 cords of brush at \$1.00 | \$1.40 |
| .08 cord of poles, at \$1.50 | .12 |
| .70 cu. yd. stone at \$1.50 | 1.05 |
| 3 lbs. No. 12 wire at $3\frac{1}{4}$ cts. | .10 |
| $5\frac{1}{2}$ lbs. $\frac{1}{4}$-inch wire strand, at 6 cts. | .33 |
| 4 lbs. $\frac{5}{16}$-in. wire strand at 5 cts. | .20 |
| $1\frac{1}{2}$ lbs. $\frac{1}{2}$-in. wire strand at 5 cts. | .08 |
| $1\frac{1}{4}$ No. $\frac{5}{16}$-in. clips at 8 cts. | .10 |
| $\frac{1}{6}$ No. $\frac{1}{2}$-in. clips at 12 cts. | .02 |
| $\frac{1}{2}$ lb. No 9 silicon bronze wire, at 20 cts. | .10 |
| Labor, including subsistence | \$1.75 to 2.00 |
| Towing | .35 to .75 |
| Miscellaneous | .10 to .25 |
| Total | \$5.70 to \$6.50 |

The cost of the upper bank revetment varies according to the amount of clearing and grading required, but rarely exceeds 10 cents per square yard. About $\frac{3}{10}$ cubic yard of stone is required to pave a square yard of bank, and the cost of the labor of placing it is from 20 to 25 cents.

The approximate cost per square yard thus becomes:

| | | |
|---|---|---|
| For grading bank | \$0.10 | 0.10 |
| $\frac{3}{10}$ cu. yd. stone at \$1.50 | .45 | .45 |
| Labor | .20 to | .25 |
| Miscellaneous | .05 to | .10 |
| Or from | .80 to | .90 |

With banks about 35 feet high above water, graded to a slope of 1 vertical to 3 horizontal and paved to the top, the cost becomes about \$11 per linear foot of bank, while the subaqueous work costs about \$18 per linear foot, as before mentioned, or a total of \$29.

*General Cost of Works on Missouri River.*—" The cost of standard pile dikes and revetments necessarily varies so widely in different situations that unit costs cannot be definitely stated. The following are the principal elements which enter the question of cost: Difference in elevation of high- and low-water planes, height of bank, distance of work from base of supplies, plant charges, extent of work and degree of concentration of same, season of the year in which carried on, condition of flow.

" For preliminary estimates $10 per linear foot is taken as the average cost of 3-row dike work and of standard revetment, including all office and incidental charges.

" In carrying on numerous works at widely separated localities it has been found that field charges are distributed about as follows: For actual construction, 67 per cent. For care, repair, and moving plant, 22 per cent. (This includes an item of but 5 per cent for light repairs.) Administration, 9 per cent. All other items, including surveys and travel, 2 per cent."* (See also p. 177.)

The itemized cost of 8748 lineal feet of revetment in 1900 was as follows:

| | |
|---|---|
| Grading bank by hydraulic process, 60,452 cu.yds. at 4.09 cts | $2,477.63 |
| Grading bank by scrapers, 1815 cu.yds. at 13.04 cts | 236.79 |
| Construction of 780,000 sq.ft. of mattress (including 4335 cords of brush at $2 per cord) at 2.61 cts. per square foot | 20,399.60 |
| Ballasting mattress and bank (including 19,800 cu.yds. of stone at about $2.00 per yard) at $1.2723 per cubic yard | 25,174.13 |
| Spalling bank (including 2918 cu.yds. of stone at about $1.00 per yard) at $1.2784 per cubic yard | 3,730.48 |
| Miscellaneous | 1,368.84 |
| Total | $53,387.47 |

This is equivalent to $6.10 per linear foot.

The total cost of general revetment work in 1910 was stated to have been from $3.25 to $8.00 per lineal foot.

*Concrete, etc.*—The cost of miscellaneous work as concrete, brushwork, etc., is given on pp. 186 to 188.

*Annual Report, Chief of Engineers, U. S. A., 1900.

# CHAPTER VI.

## LEVEES.

**History.**—Levees are embankments of earth thrown up to prevent overflow from streams, or to stop the sea from inundating adjacent lands. The Egyptians and Babylonians were the first of whom history speaks as having embanked their lands, and they were followed by the Phœnicians, the Romans, and East Indian nations. One of the earliest examples of levee building was around the city of Babylon. The Euphrates was embanked on each side, the embankments leading to the bridge across the stream. The skill displayed by the ancient Romans and others, and the extent to which they went in embanking and reclaiming marsh lands are shown in the levees along the Tiber near Rome, the Po near its mouth, and in the Fen lands in England and in Holland and other countries, but the attention of scientific men was not brought seriously to the problem until Italy began systems of levees along its rivers during the thirteenth century. The Arno, Tiber, and Po were partially embanked, followed in the seventeenth century and later by the Chiana, Adige, Reno, and many of their tributaries. The discussion brought about by the prosecution of this work resulted in a general levee system, and enlisted some of the greatest philosophers of the time, among whom were Galileo, Poleni, Torricelli, and Zendrini, and the result is that to-day these rivers are confined between embankments which, although artificial, are centuries old, and which have served as examples to Holland, Spain, France, Germany, Ireland, England, and the United States, all of whom have profited by the systematic works along the Po. The vast quantities of alluvial matter brought down by the Rhine, the Maas, and the Scheldt formed salt marshes which were later reclaimed by means of levees, known far and wide as the "Holland Dikes," and out of this marine swamp arose the rich kingdom of Holland. There are levees also along the Rhine, the Oder, the Elbe, the Vaert, and other rivers of Germany and Holland; along the Thames, Mersey, and others in England, the Loire in France, and the Vistula and Elbe in Prussia.

In the United States, while there are numerous levees in various parts of the country, and some of them of importance locally, there is but one extensive example of levee work, that of the Mississippi and its tributaries below Cairo. Prior to 1860 this important work was carried on in the States of Arkansas and Louisiana by the State governments, while in Missouri, Tennessee, and Mississippi each county bordering on the river charge of its own levees. The results were very unsatisfactory under county

governments, and, even where directed by the State, conflicting interests, lack of knowledge, and a variety of causes conspired to render the work expensive and not always of the greatest benefit. Then came the Civil War with its devastation and derangement of conditions, and the levees were virtually abandoned or wholly destroyed. Attempts were made by the local governments later on to repair them and even to build new ones, with varying success, until finally, about 1880, the Federal Government took the matter in hand. The State governments of Missouri, Arkansas, Mississippi, and Louisiana, however, continued their works in certain localities in co-operation with the United States, since which time marked improvements have been made in the design and methods of construction.

Reports show that more than $25,000,000 had been expended on levees by the United States up to 1912.*

**Location.**—Permanence, economy of construction and of maintenance, and future enlargement are involved in the location of a system of levees, but it is a rare thing to see a location made with these objects solely in view. Too often the interests of the local property holders are the first consideration. While these should be recognized to a certain extent, the general benefits to be derived from properly located lines should always be considered first. As the systems are extended and completed the flood plane will rise, necessitating new work which, if carried on with locations improperly made, will mean the expenditure of vast sums of money. It is far better to expend that money in first constructions so located as to give the best protection to the greatest number, even if such protection damages the few. In the original alignment of the levee systems along the Mississippi little attention was paid to the laws of the flow of the river; levees were located so as to protect unimportant property angles and buildings, presenting obstructions to floods at critical places without additional thickness of section. These locations have still been adhered to in many cases in the enlargements made by the General Government; in many other cases the embankments have long since gone into the river with caving banks or have been weakened at their salient angles and destroyed by the floods.

Angles should always be avoided and curves substituted flat enough to admit of a railroad track being operated. Hewson † lays down the following rules for locating a levee: "The first duty is the mapping out carefully of the bank, and, as far as may be done by a careful sketching, of the current set, the 'caving,' and the 'making.' In the case of cavings and makings, every information as to their commencement, their rate of progress inwards, and their advance downstream should be obtained carefully from local information and recorded at the proper points on the map. The cavings and the makings of the bank pass downstream in a series of waves, period after period;

* The Annual Report of the Chief of Engineers, U. S. Army, for 1911 shows the following expenditures prior to June 30, 1911: Levees, $25,735,000; revetment and contraction works, $14,046,000; dredges and dredging, $5,652,000; plant, etc., $4,055,000; miscellaneous, $12,740,000. Total, about $62,230,000.

† "Embanking Lands from River Floods."

and therefore by ascertaining the rate of descent, the rate of penetration of a 'cave,' or the extension of a 'make,' at the point of its operation, the location of the levee opposite that point may be made with a full knowledge of the conditions of its permanence." The following notes from the "Manual" of the Dutch engineer Storm-Buysing may also be quoted: "The trace of the new dike should of course have regard to economy of material and capacity for protection. Its general direction should be nearly parallel to that of the stream, but should avoid as much as possible exposure to the most destructive and prevailing winds. The fitness of the foundation should especially be regarded, and good high ground selected, free from creeks or sloughs. Sharp angles are to be condemned, especially re-entering angles, because they make basins in which the wind has full play to raise the water. When changes of direction occur they should be made by a curve, uniting the two tangents." (See Fig. 124*b*, p. 261, and Pl. 1*a*, for a general plan of part of the Mississippi levee system.)

**Section.** (See Figs. 121 to 124*b*.)—The cross-section of a levee will vary with its location, the character of the foundation upon which it is to be built, and the material of which it is to be constructed. In exposed positions, where the waves are liable to make inroads, the section of the levee must be greater and the slopes less steep than in those places where wave-action is not expected, down to solid earth. Lastly, the material of which an embankment is made will in a measure control its profile. If it is to be of sand or light porous soil it will be much more liable to be washed and cut away than if built of clay or gravel, and hence must have a greater thickness.

In this country the usual dimensions are 8 to 10 feet across the top or crown, with slopes of 3 to 1, an increase being made for levees built of sand, in which the width sometimes reaches 15 feet on top with slopes of 5 to 1 or flatter. When the levees are high a terrace or "banquette" is sometimes built on the land side for the purpose of obtaining the necessary strength. This bench is about 20 feet in width, sloping off more gradually than the main embankment, and is usually about 8 feet below top of the levee.

The levees along the Po are generally from 23 to 26 feet on top with slopes from 2 to 1, to 3 to 1, and usually have two horizontal terraces on the land side. The dimensions of course vary considerably with the conditions, in some places the top width being reduced to 16 feet. When used for roadways, a custom which is quite general there, the crown is of gravel. (See Fig. 124*a*, p. 254.)

On the Rhine the levees have a top width of about 6 or 7 feet, with slopes of 3 to 1. These narrow crowns are doubled when it is desired to use them for wagon roads. (See Fig. 124*a*.)

In Prussia the levees along the Elbe have dimensions about the same as those on the Rhine, with banquettes on the land side where the height renders additional strength necessary.

The tops of the Theiss levees rise nearly 5 feet above the highest flood-level, and are about 20 feet wide, with slopes of 2 to 1 or 3 to 1 on the river side. This slope is

sometimes flattened to 4 to 1 below high-water level, while on the land side the slope is 2 to 1 or 3 to 1. (Fig. 124, p. 252.) The land slope is broken by a terrace 13 feet wide, placed about 3 feet below high-water level. The Vistula levees in Prussia are from 12 to 16 feet in width at the crown, with slopes of 3 to 1 on the river side and 2 to 1 on the land side. There is usually a banquette having about the same width as the top of the levee.

In Holland the sections vary greatly, according to location, material, and existing conditions. They are from 16 to 25 feet on top, with slopes of $2\frac{1}{2}$ to $3\frac{1}{2}$ to 1. Those built to withstand sea-waves are of course considerably more substantial than those along the rivers, some of those in the most exposed positions having slopes of 10 to 1. (See Fig. 123, p. 251.)

In regard to section, Hewson in his work on levees before referred to, has the following: "The presentation of equal strength at all parts of the levee does not require a greater width even under the most unfavorable circumstances than (whatever may be the proper width of crown) side slopes from each side of crown at a rate of 1 foot horizontal to 1 foot vertical. Such a section may be said to be, in general, the section of uniform strength. The strength of anything being the strength of its weakest part, an excess of strength at any one part is, it is almost needless to observe, a waste of material in leveeing, and consequently a waste of money.

"In practice, however, it is impossible to conform to the section of uniform strength in levees, seeing that the controlling consideration rests in the standing angle of the material. The standing angle of clay has been set down at 8 inches base to 1 foot in height; and, therefore, may be held to conform closely to the section of perfect economy of material—the section of uniform strength. Twenty-one inches of base for every 12 inches of height being the standing slope of sand, that material is seen in the excess of its natural section over the section of equality of strength to involve in leveeing a very large waste of material, and, therefore, of money. In a levee having a 3-foot crown, a 21-foot base, and a height of 5.2 feet, the area of cross-section is 62.4 square feet. This levee, it must be recollected, is one of equal strength; and, therefore, measuring its effective strength by its weakest part—its 3-foot crown—we find the limit of its actual resistance to be, when made of sand, as 3 feet$\times$95 pounds, or 285. A clay levee of 2.11 feet crown, sloped down at the standing angle of clay to a base of 9 feet for 2.5 feet in height, contains within it the slope of uniform strength, and consequently its crown being its weakest part, the limit of its effective resistance is as $2.11 \times 135$, or 284.9. This clay levee of 2-feet crown and 9-feet base presents, then, precisely the same resistance to water-pressure as does the sand levee of the same height, having a crown of 3 feet and a base of 21 feet. The cross-section of the clay bank in this case is 29 square feet; while, as has been said above, that of the sand is 62 square feet. But practice goes still further in increasing this disproportion between the different quantities necessary in levees of sand and in corresponding levees of clay. The standing angle, as presented in theory, must be deviated from in both sand and

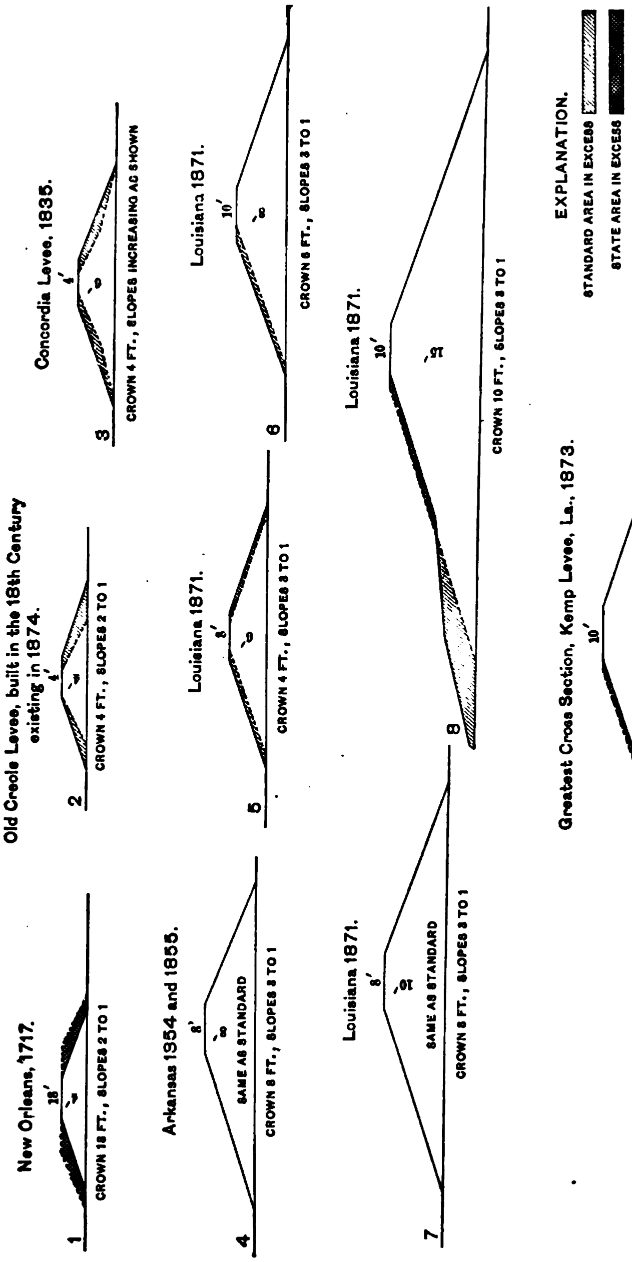



FIG. 121.—Sections of Old Levees, Mississippi River.

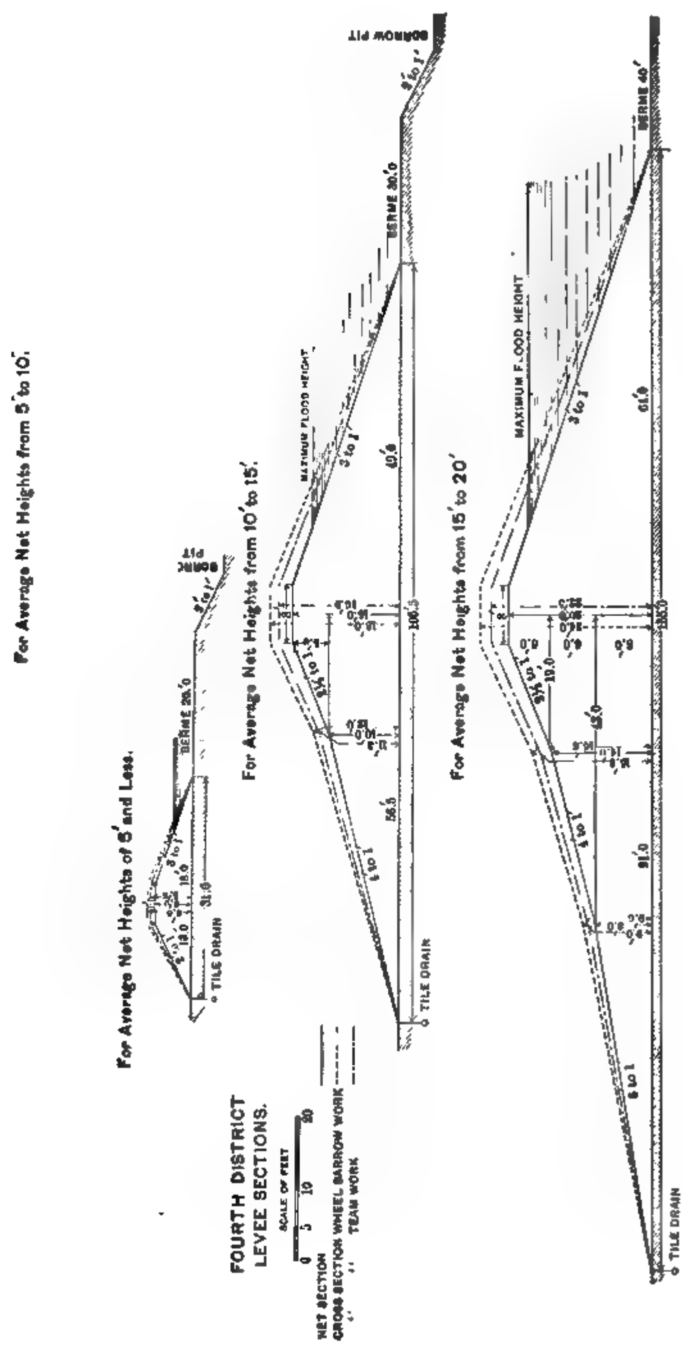



FIG. 122.—Sections of Mississippi River Levees for Various Heights.

clay in order to meet, in practice, the contingencies of floods and rains. Lighter, looser, and less adhesive than clay, the flattening of slopes in sand below that of the angle or slope of repose must be much more considerable in practice than that in the heavy concreted and adhesive bank of clay, in order to resist without endangering the effective strength or stability of the bank the active washes of rains and waves. . . Confining the equation of the materials to the simple fact of the difference between their strict standing angles, 26 yards of clay are seen to be equal, in a levee of 5 feet height, to 58 yards of sand, in accomplishing the object of all levees, namely, effective resistance to floods."

In regard to the slopes given to levees in America (see Figs. 121 and 122), another author * states that " the slopes of 3 to 1 now generally adopted are the outcome of a long experience. In the early stages of levee building, much smaller dimensions were often used, but disaster was frequently the result. A steep back slope very often leads to extensive sloughing or slipping, especially if the material be at all weak. A steep front slope exposes the embankment to serious abrasion by waves. A slope of 2 to 1 is less than the angle of repose of wet earth of almost any kind. In very small levees, where the width of the crown cuts an important figure, slopes may often be reduced. Thus a levee 3 feet high, with a crown of 8 feet and slopes of 2 to 1, has a stronger section than a levee 12 feet high with slopes of 3 to 1.

" Where the exposure to winds is very great the front slope is often made as flat as 5 to 1, the back slope being then reduced to 2 or 2.5 to 1. It is found that a flat slope is a great protection against the wash of waves, and that a well-sodded ' buckshot ' levee, with a slope of 5 to 1, will stand a pretty stiff wind. If the sod be once cut through, however, and a hole made in the clay, the latter is liable to be undermined, and the superincumbent masses of earth fall in huge blocks."

The following gives information as to the sections generally used:†

" A few years ago the Government adopted standard sections which have since been adhered to, except in cases where the conditions demanded more specific treatment. The first, second, and third districts of the Mississippi River have practically the same standard for all levees on ordinarily good foundations, and when constructed of material not below the average in strength.

" The standard dimensions are: Crown, 8 feet; front or river slope, 3 to 1; back slope, 3 to 1. Where the levee is over 11 feet in height, a banquette, at an elevation of 8 feet below the top of the main levee, is added. The slope of the crown of this banquette is 10 to 1, width of crown 20 feet, and back slope 4 to 1. Where the foundation is bad, or the material weak, the banquette section, and perhaps the front slope of the main levee, is increased.

" The specifications require the levee to be constructed in 2-foot layers, with scrapers, on a well-grubbed and thoroughly plowed foundation containing a small exploration

* "Levees of the Mississippi," Wm. Starling.

† "Standard Levee Sections," H. Coppée.

muck-ditch filled back with strong material, the best to be found in the vicinity, and sodded at 2-foot intervals with Bermuda grass.

"In the fourth district the dimensions of the standard adopted vary with the height, and are intended to conform more nearly to the supposed theoretically perfect section. These variations may be further modified, as in the other districts, when required by abnormal condition of foundation, material of construction, wave wash, etc. (See Fig. 122.)

"For levees from 5 to 10 feet in height, the crown is 8 feet, the river slope is 3 to 1, and the land slope $2\frac{1}{2}$ to 1.

"For levees from 10 to 15 feet in height the crown is 8 feet, the river slope is 3 to 1, and the land slope 4 to 1 to within 5 feet of the crown; thence to the crown it is $2\frac{1}{2}$ to 1.

"For levees from 15 to 20 feet in height the crown is 8 feet, the river slope is 3 to 1, the first 8 feet of the land slope from the ground is 6 to 1, the next 6 feet 4 to 1, and thence to the crown $2\frac{1}{2}$ to 1.

"In the upper districts 10 per cent of the height, both in wheelbarrow and team work, is required for shrinkage.

"These standard sections are expected to withstand the water to within 3 feet of the crown of the levee, without excessive saturation or change of form, and to give unqualified protection under all normal conditions of foundation and materials of construction.

"When subjected to water above the 3-foot line, though they are intended to remain intact, they cannot be considered, either theoretically or practically, standards of excellence.

"Without taking into account the effect of waves on exposed levees, which necessitates recourse to special slopes and methods of protection, planking, revetments, etc., the whole question of standard section depends on the permeability of the embankment and foundation, that is, the extent of seepage, or percolation, and the best form and method for overcoming it in different materials.

"In 'buckshot' or clay, which is practically impermeable, the section might be given a strictly theoretical form, dependent alone on the height of the water and the weight of the buckshot; allowing some crown merely for increasing the height in time of excessive flood, the slopes being plane surfaces with an inclination sufficient to insure the required weight to counteract the hydrostatic pressure and the angle of repose of the material.

"In cases of permeable materials, light clays, sand, and loam, the levee becomes partly saturated when subjected to high water, the line of demarkation between saturated and dry soil descending in a hydraulic gradient varying in inclination with the soil of which the levee is composed, and being probably very irregular in trace because of the lack of homogeneity of the material in the body of the levee.

"In surface soils, subject to direct rainfall or percolation, from adjacent watered

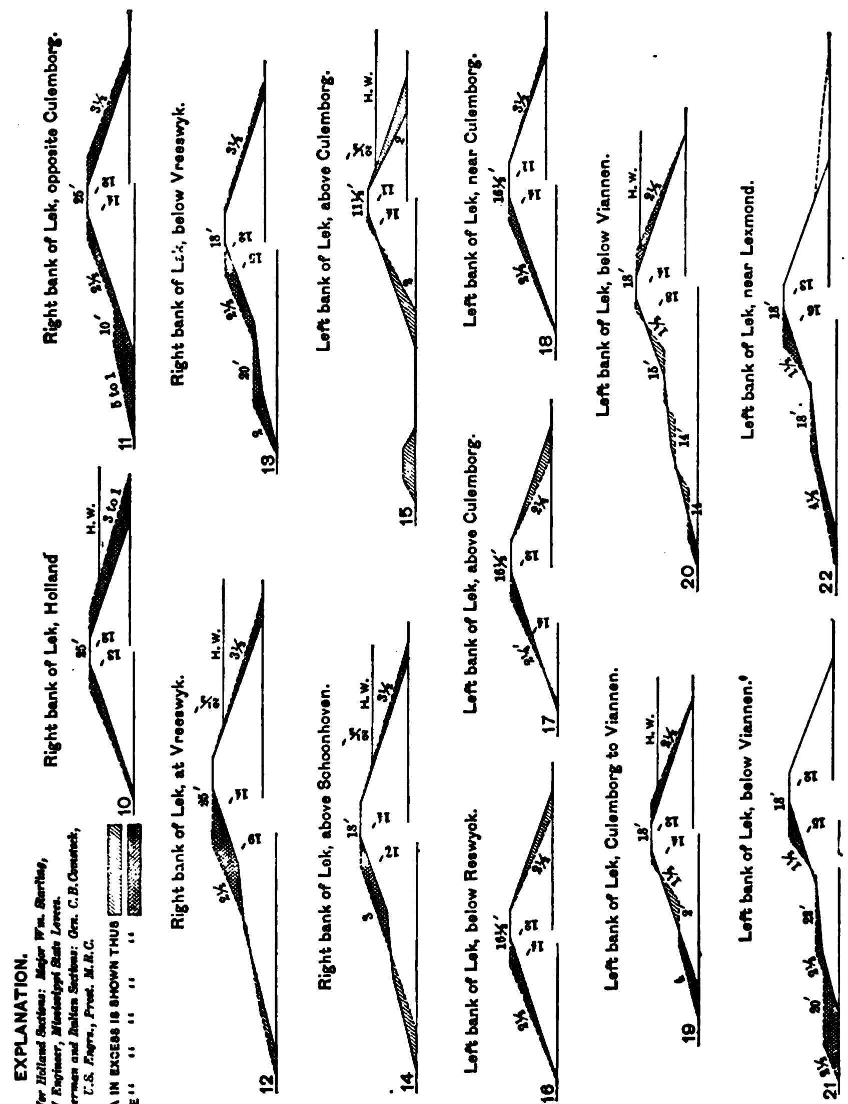



FIG. 123.—Sections of Levees in Holland.

Section in 1850.

Section in 1880.

Section above Csongrad in 1888.

Section below Csongrad in 1888.

Maximum Section in 1895.

FIG. 124.—Sections of Levees in Hungary. From Report of Chief Engineer, E. von Kvassay, 1908.

areas, the ground-water stands at a level dependent on the composition of the soil, both physical and chemical, the natural and artificial voids, and the hydrostatic pressure. In nearly all soils, remote from intersecting fissures, wells, or streams, the line or plane of saturation is parallel with the surface of the ground, following the inclination of hill and valley. Where wells, fissures, or river-beds occur in the surface soil, the line of moist material, or plane of upper surface of saturation, is inclined towards the fissure, well, or river, the degree of inclination depending on the consistency of the soil.

"The power of soils to resist the pressure of water is due to their specific gravity, fineness of comminution, cohesiveness, and the irregularity of individual particles. Coarse sharp sand has greater resisting power than that composed of fine, smooth, rounded particles.

"The author estimates the strength of materials, as found in this levee district, to resist deformation due to seepage, or their value for levee purposes, in about the following order:

"(1) Buckshot and gravel tamped in shallow layers.

"(2) Buckshot artificially mixed with sharp sand in shallow layers.

"(3) Buckshot or clay.

"(4) Heavy strong soils.

"(5) Coarse sharp sand.

"(6) Light soils.

"(7) Fine sand, rounded particles."

**Height.**—One of the principal causes of breaks in levees is their insufficient height. No embankment of earth, and particularly of the class of earth of which levees are generally constructed, can long withstand the action of water flowing over its crest. It is, therefore, of the utmost importance to build to a height which is not liable to be reached by the greatest floods. This elevation is difficult to determine in advance, because of the uncertainties attending the coming of floods and the additional height to which the river may rise, because of the contraction caused by the levees themselves. A lack of funds has restrained the engineers on the Mississippi from building to a safe height for great floods, and they have been compelled to adopt what is called provisional grades, that is, grades adjusted to the resources. Those adopted in recent years have been as follows: Third district, 3 feet above highest flood; fourth district, 2 feet 6 inches above same; Upper Yazoo district, 4 feet above flood of 1890; Lower Yazoo district, 4 feet above flood of 1891; St. Francis district, 3 feet above flood of 1882. Recent floods have established better standards for height than were previously available.

The effects of settlement and sloughing at various points have rendered this grade line uncertain, and in flood times it has been found necessary to add to the heights in many places in order to save the levees from destruction by overflow. The establishment of a grade which would not be reached by a river wholly confined by levees has

the Mississippi are known generally under the name of "Buckshot," the name being given because of its peculiarity of breaking into fragments about the size of bullets. Buckshot is not wholly clay, there being a varying percentage of sand in its make-up. The sand used in levee construction is mixed with earth and is of fine particles. The clay is usually plastic and of the blue variety, although other kinds are found. The loams are composed largely of sand, and are light and fine, and very poorly suited to embankment building. By reason of its resistance to the action of the waves, as well as to percolation, clay is the best material found for this work.

Sand makes a fairly good embankment when not subjected to wave action, but it is not safe to build one of small thickness, because a cavity once formed will increase rapidly and soon endanger the whole structure. In fact, sandy material of any kind is unreliable and difficult to manage, and even when covered with sod is not to be depended on. Where it is practicable to place a layer of clay upon the outside fairly good work can be obtained, but when a better material can be had without too great expense the use of sand is inadvisable. Not only is the wave action severe on sand embankments, but they are liable to sink and slough as the fine particles are washed out, and once sand starts to escape it is a difficult matter to arrest its movement. Whole embankments will thus sink away without warning and with great rapidity. Clay will resist water and erosion to a much greater extent than sand, but it does not stand at grade very well. Its tendency is to settle and crack, and in this it is inferior to good sand. To make a reliable embankment it must be placed in thin layers and be well tamped as put in. It is very difficult to handle in wet weather and gets very hard during a dry season.

The fineness and lightness of loam renders it undesirable for levee work. Its particles lack coherence, and it is even more treacherous than sand, because saturation transforms it almost into mud. Where loam is used the embankments should be given much larger dimensions than with those of clay. On the other hand, it is usually very abundant, being a surface soil, and it is easily worked in dry weather, packs well, and holds up to grade.

A combination of clay and sand is highly recommended by most engineers who have had experience in levee construction. It gives a bank of a permanent nature and is also easily built. It prevents the cracking noticeable in clay embankments and does not shrink so much, and at the same time the tough quality of the clay is preserved.

One of the most important materials used for river embankment is gravel, which in many localities is easily procured. Its use is generally as a facing to prevent wash, the inner portions of the levee being of a more cohesive material.

Upon this subject Hewson says: "The lightness of a sand-bank is but a small disqualification for leveeing compared with its liability to wash and leak. Its wash is not confined to waves, current, and rain; but is carried on actively by the wind. Sand is liable not only to run and blow away in a dry state, but also in a wet state is liable

to run or 'melt' like so much sugar. But while its lightness lays it open as a material for levees to great objection on the ground of duration, the worst of its properties in such works is its liability to percolation. A bank which may be of ample section to resist the total pressure brought to bear on it, when that pressure acts from the outside slope against the whole weight of the bank, will yield when that pressure becomes transferred from the outside of the bank to some point or plane within it. In the latter case a portion only of the whole mass is engaged in the resistance of the whole pressure. Now percolation of the water into the body of the work places the levee under these very circumstances.

"A thread or plane of water finding its way into the interior of an embankment exerts just as much pressure against the earth on each side of it as if that thread or plane were an ocean of the same depth as that thread or plane. As this thread separates the parts of the levee the outside water fills up the split, and by thus preserving the same height of water within the split as at the beginning of rupture, the levee becomes completely rent asunder, and thus reduced in its aggregate power of resistance is finally swept away. Porous materials, then, in water-banks, no matter what be their weight in the banks, tend by the insinuation of water-threads between their parts to the destruction of those banks—this tendency, however, being greatest at the time of the construction of the works, and least at the time when their adhesion shall have been perfected by the coating of deposit over their external faces, and the insinuation by filtration in their internal pores of earthy matter.

"Loam is much better for water-banks than sand. Thirty per cent heavier, it meets all the conditions involved in leveeing on the ground of weight much better than sand. Much stancher in its parts, it is superior to sand in all those serious objections applying to sand for the purposes of watertight embankments. The very best of those soils obtainable under the present practice on the Mississippi for the purpose of river banks is blue clay. Several kinds of this clay are found on the lines of the levee works, but they are all subject to the disadvantage of a greater or less admixture of fine sand. Perfectly impervious to water as they all are, the presence of sand lowers their usefulness partly by involving a lighter weight, but mainly, and sometimes even to a very serious extent, by giving them a tendency, especially after frosts, to melt or run like marl in water. But notwithstanding these drawbacks the clays of the Mississippi bottom furnish its very best material for leveeing."

There is much in the class of material employed, without doubt, but there is also need to place this material in a careful, proper manner. The best of materials will not give satisfactory results if thrown carelessly into an embankment, while an inferior grade of earth may be so built into a levee as to make a safe and permanent bank. Sound earth is a good enough material for levees, but the bank must be carefully built, of sufficient dimensions, and, especially with a light or treacherous subsoil, must have its base extended by a banquette.

The opinions of the Dutch engineers, who are probably the leading authorities of the world on the construction of levees, are worthy of attention in connection with this subject:* "The earth of which the dike is to be composed must be such as to cohere readily with itself and with the soil beneath it. The more cohesion the soil has, the more it is to be preferred; and the more will its different parts unite and form a compact mass which can oppose resistance to the water, and thus furnish a tighter dike. Clay is thus the most suitable earth for dikes, and for the most part is to be found along our coasts, where dikes are to be built. It is to be procured, by preference, from the outer side, but when the fore-shore is scanty or wanting, it must be taken from the land side. Sand has very little coherency, and does not afford a watertight and strong dike. Peat and swamp soil have too little specific gravity, often less than water itself, and must thus, as well as sand, be rejected from the dike. Mould or arable land, though far inferior to clay, is still much better than peat or sand, packs closely, by reason of the smallness of its particles, and is especially suitable for dressing slopes that are to be sodded, as grass grows very well upon it. Clay cannot always be had in a pure state or in sufficient quantities, so that inferior earths must sometimes be mixed with it. But if the precaution be taken to work the best and purest clay on and near the outside, and the inferior sorts in the body of the dike, such sorts may be used without great danger. There are examples of dikes that consist of very sandy soil and have a covering of only one meter of clay on the outer slope, yet they furnish very satisfactory dams. It is easy to be seen, however, that such a dressing of clay must be treated with the utmost care, and the slightest injury to the outer slope must be immediately repaired, for if enough of the clay be removed to permit the water to come in contact with the sand or peat, very little confidence can be placed in the dike."

**Construction.**—As has been stated, great importance must be attached to the care with which embankments for water are built. If made simply, as those for railroads are constructed, they will be more or less permeable, and when the water comes against them settlement and deformation will result. It is necessary, then, that precautions be taken in cleaning up the foundation and in rolling or tamping the material in place so as to make it compact and close-grained.

Before beginning the construction of a levee it is important that all vegetable matter, trees, etc., be removed from the site. This should include the roots as well as the trunks and branches. After a thorough cleaning the ground should be plowed or spaded deeply in order to secure a more perfect bond with the new structure, and if the top soil is unsuitable it should be removed before bringing on new earth. Generally the specifications provide for cutting a muck-ditch near the center line of the levee, at the discretion of the engineer. This ditch and all excavations made in removing stumps, trees, etc., are filled up with approved material, well tamped.

The following clause from Government specifications gives the method of building:

* "Holland Dikes;" Starling. Am. Soc. C.E., vol. xxvi, p. 694.

" The embankment will be started full out to the side-stakes, and be carried regularly up to gross fill, in layers not exceeding 2 feet in thickness, when built by scrapers, and 6 inches when built by wheelbarrows. In wheelbarrow work the earth will be carefully tamped either by wheeling over the embankment or by employing one rammer to two wheelbarrows. When the embankment has been brought up to the proper height, it shall be dressed, and planted with living tufts of Bermuda grass, 4 inches square, and not more than 2 feet apart, well pressed into the earth and lightly covered with soil, to the satisfaction of the engineer in charge, or his designated agent. The contractor will cut down all trees, both great and small, to a distance of 100 feet from the base of the levee on both sides.

" Only clean, unfrozen earth, free from all foreign matter, shall be used in constructing the embankment. It will be procured on the river side generally. In no case must it be obtained within 40 feet of the base of the levee on the river side, or within 100 feet on the land side, and the side slope of the pit next to the embankment not to be steeper than 1 on 2. At intervals traverses must be left across the borrow-pits to prevent the flow of a current along the levee. The distances between the traverses will not be more than 500 feet. They shall be at least 10 feet wide on top, with slopes of 1 on 2. Borrow-pits must not exceed 3 feet in depth on the side next to the levees, but they may gradually deepen with a slope of 1 on 50 when on the river side, and 1 on 100 when on the land side of the levee. All existing levees, or parts of old levee, must be left, unless written permission of the engineer in charge is given for their removal.

In connection with methods of construction Mr. Starling says:* " Preference is generally given to the wheeled scrapers, especially if it is expected that water shall get against the new levee immediately. Generally, if it be possible, the levee is built a year or so earlier than it will probably be needed, in order to give it time to settle thoroughly, and to be completely covered with sod. With certain kinds of soil there is an objection to scraper-built levees, namely, that they are very liable to be cut and washed into gullies by rain before the sod has had time to grow. These soils are loam and mixed sand. When put up with wheelbarrows, banks of such material are at first comparatively loose and porous, and absorb water like a sponge. By the time they have settled fully, the sod has grown. When the earth, however, has been put up with scrapers it is very hard, and sheds water like the roof of a house. The material being light and friable, however, the rain soon cuts channels which it uses regularly, and the gullies which are the result of this action sometimes cut almost through the crown of the levee. A slope thus eroded has to be re-dressed before it is sodded, and the new dressing is liable to be washed away also. In spite of this objection scraper-built levees are generally preferred, and some engineers place such restrictions on wheelbarrow work as almost to prohibit it. The shrinkage exacted is generally one-fifth for wheelbarrow work untamped, or one-tenth if it be tamped, and one-tenth for scraper work."

* " Levees of the Mississippi," p. 8.

Heretofore the construction of levees has been carried out almost entirely by hand methods, owing probably to the remoteness of the sites, and the difficulties attendant on the use of machinery in such work. Recently, however, steam dredges have been employed with success sufficient to demonstrate that by the use of suitable machines, satisfactory construction can be obtained, and at a great reduction of cost.* To obtain economical results the dredges would be used as much as possible while the river was at bank stage, but would also be provided with pumps by which the borrow-pits, in which they were digging, could be flooded so as to keep them afloat. The wet material cannot be pyramided to the full height, and, in order to avoid loss by sloughing the dredge works several times over the line, adding to the material in place at each working after the material already put in has dried out.

**Muck-ditch.**—Mention has been made of a trench excavated along the line of the levee near the center of its base. This is called a muck-ditch, and it has a double purpose in that it is an excellent way of investigating the character of foundation soil, and furnishes, when filled with selected earth and tamped, a diaphragm through which the water will not seep. This feature in levees has caused much discussion among engineers, some claiming that it is of little value, or is even hurtful, while others contend that it is a useful and even necessary precaution against destruction.† Mr. Starling says: "Generally levee engineers, while they recognize that muck-ditches do a certain amount of good, as they have very limited means, prefer to spend their money above ground. They therefore give the ditches small dimensions, using them, in fact, rather for exploration than for any other purpose."

Mr. Hewson says: "The practice of cutting out a trench for the puddle, or 'muck-ditch' as it is called on the Mississippi, in the natural surface of the ground is generally useless, and sometimes positively mischievous. When retentive subsoils exist under the base of the proposed bank, then it is certainly a clear gain in stanchness to run down the puddle-wall of the levee to a bond with the underlying impervious earth. But the experience along the shores of the Mississippi leads to the presumption that, in those cases where the sand does not commence on the surface, a ditch of 3 feet deep is more likely to present a bottom of sand than of loam or clay. The rationale of those muck-ditches rests on their usefulness in preventing leakage; and therefore, supposing the ditch and wall carried up regularly with a puddle, those ditches in a great majority of cases failing to reach a more retentive soil than at the surface of the ground, involve in all those cases an utterly resultless waste of money. Besides, to undertake to prevent leakage through the porous earths of the natural shores of the river is a hopeless labor; and so far as the strength and durability of the levees are concerned is a labor also perfectly useless. It accomplishes nothing whatever for the artificial embankment. But in some cases these muck-ditches are, as already stated, mis-

* Annual Report Chief of Engineers, U. S. A., 1899, p. 3539.

† See Chapter X, "The Colorado River and The Salton Basin," 5th paragraph.

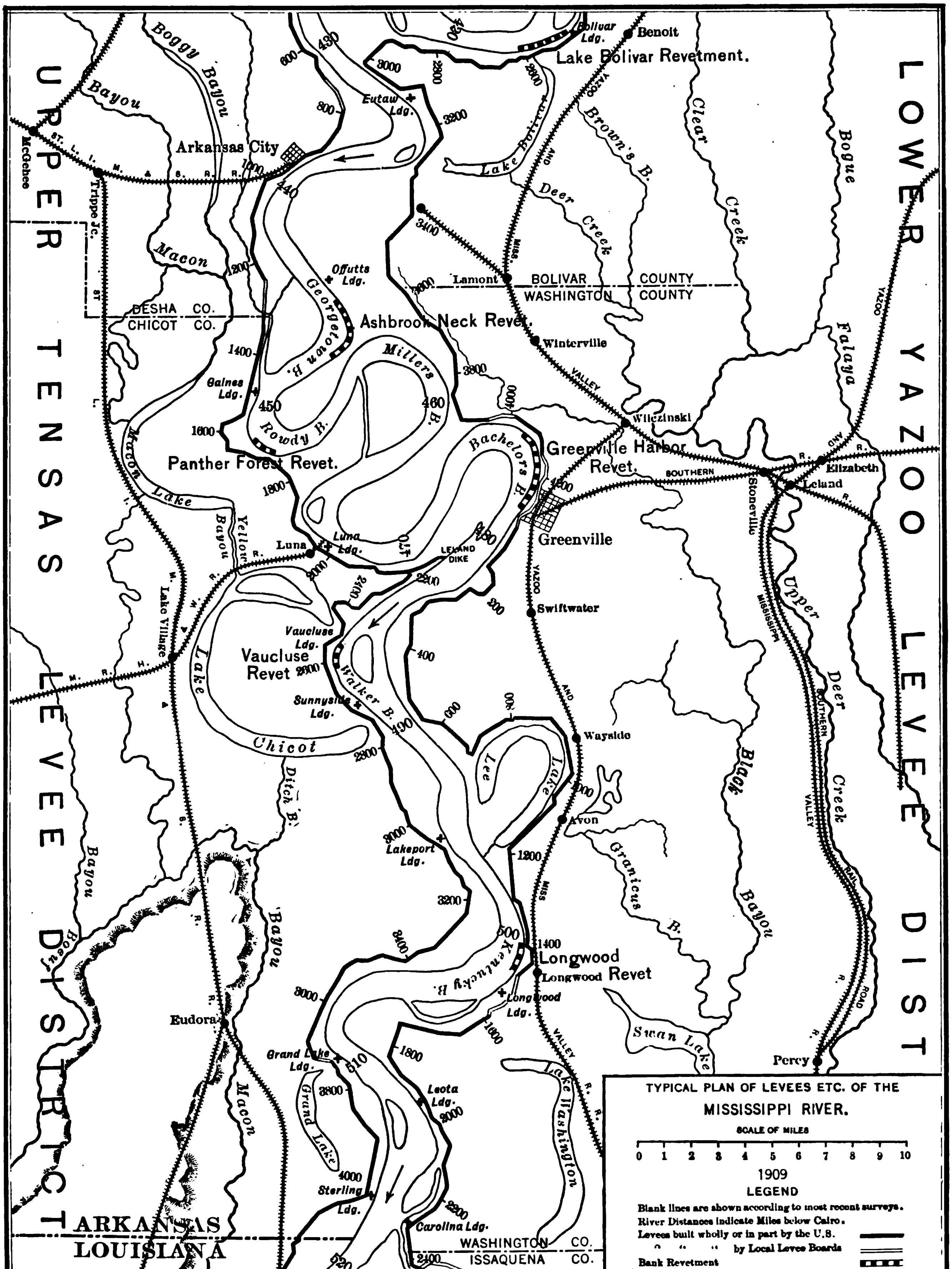

TYPICAL PLAN OF LEVEES ETC. OF THE
MISSISSIPPI RIVER.
SCALE OF MILES
0 1 2 3 4 5 6 7 8 9 10
1909
LEGEND
Blank lines are shown according to most recent surveys.
River Distances indicate Miles below Cairo.
Levees built wholly or in part by the U.S.
" " " by Local Levee Boards
Bank Revetment
UPPER TENSAS LEVEE DISTRICT
LOWER YAZOO LEVEE DIST
Arkansas City
Lake Bolivar Revetment.
Ashbrook Neck Revet.
Panther Forest Revet.
Greenville Harbor Revet.
Greenville
Vaucluse Revet
Longwood Revet
BOLIVAR COUNTY
WASHINGTON COUNTY
DESHA CO.
CHICOT CO.
WASHINGTON CO.
ISSAQUENA CO.
ARKANSAS
LOUISIANA

FIG. 124*b*.

chievous. Across those lagoons or creeks which are dry during periods of low water the foundation for banks consists generally of a hard crust of clay for a few feet thick, overlying quicksands or thin puddles. These crusts, like the grillage of timbers used for the foundations of some engineering works, are highly valuable in those situations, by diffusing the weight of the superincumbent levee over a wide bearing; and thus, though unequally loaded by the necessary cross-section of the levee, assist, in proportion to their strength, in distributing that bearing equally. This, where not sufficient to obviate the sinkage altogether, reduces it considerably; and in bringing a large area to act in the resistance, assists in guaranteeing, with the least possible sinkage, and therefore the least possible loss of work and money—a finally well-sustained foundation. The muck-ditch, however, cuts this natural platform for the levee in two parts; and over this cut, the greatest weight, that at the crown, pressing vertically, acts as with a leverage in bending down, and finally breaking off, the natural crust of the surface. The necessity therefore follows, under those circumstances, of employing an excess of earth in forcing out laterally, and forcing down vertically, the running sand or soft puddle of the underlying foundation in order to compress those soft materials into a compactness sufficient to present an effective resistance to the weight of the superincumbent embankment."

**Banquettes.**—As has been stated, in order to strengthen high levees a terrace is generally built along the land side. This is called a banquette. Its dimensions vary considerably in different localities, its crown at some places being about 20 feet in width, while at others it is twice that amount. It has a very flat slope and usually commences about 8 feet below the top of the levee, and is quite frequently used as a road.

Levee engineers claim that banquettes should be built at all points where the foundation or material of the levee is weak, and behind all embankments having a height of more than 12 feet, regardless of foundation or material, in order to have working room and material in time of extreme high water. They should be built with the levee and not as future enlargement, all later enlargements being made on the river side to cover weak spots in the old levee or its berme and to prevent sloughing.

In addition to strengthening the levee itself, the banquette is an excellent means of re-enforcing the natural ground underneath. This is frequently porous, light, and more or less traversed by roots, etc., and may give way to the pressure from without which will " blow up " the ground inside the levee. By building on this a bench of earth the hydraulic head is diminished.

The practice of using the banquette as a roadway is not a good one, but as those interested in levees would otherwise have to provide highways at considerable cost it is not practicable to wholly exclude travel. On some of those, where the crown width is about 40 feet, a space of 15 feet next the levee is fenced, the road placed upon the remainder. In this way the levee is protected and the public accommodated. Some time since the authorities in charge of levee-building decided to expend no money in

improving embankments used as public highways, since which time a marked decrease in this use has been noted, and other roads are being built. In those localities, however, which are many, where it would be impracticable to get a new location within a reasonable cost, the levees are still used.

Although a roadway along a levee is without doubt injurious in that it allows stock, particularly hogs, to tear it up, and otherwise leads to its damage, there are yet some very good advantages to be derived from it. It facilitates general inspection and weed-cutting, and, in fact, all work pertaining to the construction and maintenance of the levee system. If travel be excluded in flood times there is not serious danger to the levee. Even without roadways hogs find their way onto the levees and do much damage.

**Protection.**—When a levee of any height has been completed it should be well sodded, where the climate is suitable, with tufts of Bermuda grass. This is the simplest and most useful of revetments; it throws out lateral runners and rapidly covers the surface, and grows well in exposed or sheltered situations. After it is once started it will withstand drouth, freezing, and floods, and affords an excellent protection from rains, and even from wave wash, where the soil is strong. For a long time there was such strong prejudice against it that its use was excluded, but now it is included in the levee specifications, being placed in tufts 4 inches square set 2 feet apart.

The growth of grass soon mats the covering together and makes a rather durable protection, and on the land side it is preferable to apply nothing else because it is important to be able to see the first signs of degradation. On the river slope at exposed points, flat stones (water-wings) are employed; but their expense is too great for the custom to be widely applied, and herbaceous or bushy vegetation is more general. Trees with high trunks and heavy roots must be excluded, as they may become dangerous to the levee.

A recent Government report says that the high water "has demonstrated very clearly the value of a good sod which holds the slope in place, preventing sloughing even when the levee is saturated; also, the fact that 1 to 3 slopes, when well sodded, will withstand wave wash."

In cases of extreme exposure pile-work and broken stone are much employed, placed both parallel to the levee and in the form of spur-dikes. Gravel is also a good protection where the currents are not too strong. The protection of embankments by quickly growing grasses is very general in Europe. Willows and other forms of small trees are also planted for such purposes on the river slope, thus holding the earth by their roots and breaking the wave-current by their branches. In some countries straw is used, twisted into strong ropes, which are laid along the levee side by side and held down by stakes.

**Maintenance.**—The maintenance of a levee includes all measures necessary for its protection and welfare, not only when it is threatened, but at all seasons. The cutting of weeds and repairs of injuries by stock are as important as those of greater magnitude during flood times. The measures for high-water protection are often, of necessity,

crude and of a temporary character. Usually they are carried out under the direction of proper authority, but there are times when the population must take matters into their own hands and hastily improvise such works as will protect their interests.

Experience has shown that when levees break it is ordinarily from one of the following causes:

(1) Insufficient height.
(2) Leakage.
(3) Sloughing.
(4) Wave wash.
(5) Cutting.

(1) It is evident that a levee which is low enough to permit the water to flow over its crown will, under ordinary conditions, be destroyed unless protection is promptly given. At first the water merely trickles down the embankment, causing a slight wash; but this rapidly enlarges as the flow increases, until, finally, the waterfall pours through the cut and tears out large masses of earth. Once well started it is difficult, if not impossible, to stop it, so that it is far better to apply the remedy in advance of the real danger.

The prevention of a break from this cause consists in raising the grade of the embankment by means of earth taken from the most convenient point and placed upon the crown. Too often this point is the body of the levee or its banquette, because everything else is under water at the time. Much of this raising of height is done with teams and scrapers, but as the addition grows in height it also decreases in width, so that it may be necessary to complete it with wheelbarrows. Another common method of increasing the height is to set posts or drive stakes into the front edge of the crown and against these set up planks. This structure is then backed up with earth. Probably this is the least expensive and quickest method of increasing levee height. Another method largely resorted to is the placing of sacks filled with earth on the top of the levee, sometimes in single tiers, sometimes piled to considerable height. They are backed up with earth if considered necessary.

(2) Leakage may arise from several causes, but one of the most common is the crayfish. Burrowing animals of several kinds also work holes through or under a levee, and once the water is started through these openings it rapidly cuts away the embankment until it is checked. It is not probable, however, that all holes can be ascribed to these causes, and part of them at least are chargeable to the decay of roots in the ground. Whatever be their origin it is necessary to look after them promptly when they begin to discharge muddy water, for then it is evident that erosion is taking place within the body of the embankment, and that the opening is being enlarged.

Even when levees are perfect in themselves there is often a continual transpiration of water through the foundation soil during flood stages. This water, called "seep-water," is not only injurious to the lands within the inclosed basin, but may also become

dangerous to the stability of the levee, should the base be somewhat narrow. Hence the necessity for the banquette. It is not unusual to see the water break out through the soil inside the levee and throw up considerable mounds of sand. These eruptions are locally known as sand-boils, and as a general thing cease to throw out anything but water after a short time. However, during a flood all these leaks, of whatever nature or origin, may become quite dangerous, and then it is necessary to arrest their flow. The method in most common use for this purpose, both in this country and abroad, is to surround them with a small levee, the ends of which are joined to the main embankment. This is called " hooping." This little bank is usually built to a height about 2 feet below the river level, thus relieving the levee of a portion of the strain and reducing the erosive action within.

In regard to " sand-boils," Mr. Starling says:* " Sand-boils are very common and very alarming incidents of every high water, and have been especially prevalent since the great accessions which have been made in recent years to the height of levees. At their first occurrence a stream of water suddenly bursts through the ground, throwing out volumes of sand of several cubic feet, or perhaps even of yards, which it distributes around the circumference of the hole. Sometimes large numbers of these outbursts occur in the same locality; indeed there are some situations which are peculiarly liable to them. Twenty or thirty will be found in a length of 200 or 300 feet. They do not usually occur very near the base of the levee, but 40 or 50, 100 or 200 feet away; indeed they are occasionally found 1000 feet distant from the levee. . . . When the boil is near the levee and of considerable dimensions, it is unquestionably a symptom of danger. Perhaps a little more, and it would have been a crevasse. . . . When the manifestation is formidable, the remedy generally applied is to weigh down the weak place with an additional load of earth, which is usually laid upon a thick layer of brush, to allow a partial drainage of the leak water and to prevent the contamination of the new and dry earth by the mud of the old."

(3) One of the most alarming experiences of the levee engineer is that which comes when, with the water rising upon the outside to nearly the full height of the levee, he sees the land slope sloughing away from the excess of permeability of the embankment. The chief cause of sloughing may be laid to a lack of proper care in construction. Had the earth, during the building of the levee, been well tamped in layers, saturation would have been less liable to occur, and their rupture might have been avoided. A French authority † has thus described the process.

" In the degree that the water rises in the river, oozings of increasing extent are noticed on the opposite side of the levee, and where the maximum of permeability exists (taking account of the pressure and thickness), the slope on the inner side is weakened and a first subsidence is produced. This subsidence, often small in itself, is followed by others which come more rapidly as the permeability is increased by the diminution

* " Levees of Mississippi," p. 13.

† " Rivières à courant libre;" De Mas.

of the thickness. Then, after a continuance of these slips, the rest of the bank, having no longer a sufficient resistance, is carried away and the break enlarges rapidly. It is seen that the rupture does not occur suddenly under the influence of saturation; the dike melts and is liquefied like a morsel of sugar in contact with water; it spreads out on the side exposed to the air and only gives way finally when these seepages have sufficiently weakened it. The damage may, in fact, be attributed to a double cause; first, the slope which too light earth takes when dry is not sufficient for equilibrium when it is wet; secondly, that this earth is carried away by infiltration."

In order to prevent this seepage and sloughing several solutions have been offered and tried. In France, a stanch revetment of stone paving has been placed along the river side of the levee to prevent contact with the water. This is very expensive, and has led to much criticism, not only on account of the cost, but also because, it is claimed, breaks in the paving will occur and allow the water to come against the bank, thus destroying its effect. Another solution lies in giving the inner face the slope which its material would take when wet, but as this does not prevent the removal of material by infiltration it has been proposed to wall this inner slope with dry stone in order to prevent this escapement. This has been done in the protection of cities, but is too expensive to secure general application.

The high waters of recent years in the Mississippi valley have demonstrated pretty thoroughly the necessity for a complete drainage of levees in order to overcome this difficulty. Ditches of sufficient capacity to carry away all seep-water are cut along the base of the embankments, and little ditches are made down the slope into them. The latter lead the waters off as rapidly as they percolate through the pores. This prevents the softening of the material and the sliding which would otherwise take place. This practice is becoming pretty general and has proved quite effective, judging by the reports of engineers who have resorted to it. When the sloughing has been started it may be checked by driving heavy stakes well into the embankment and bracing them back to similar stakes. In front of the first row of stakes, brush, saplings, or any convenient wood must be laid horizontally to form a barrier for the earth to lodge against. Brush is also placed upon the portion sloughed and covered with earth until the normal levee section is practically restored.

Another method of the treatment of sloughing levees is quoted from a Government report: "Treatment of sloughs is simple if the principle involved be understood. It is usually the case if one occurs where an inexperienced man is at work, that he will endeavor to restore the lost section with earth or sacks. This does no good, but rather tends to augment the trouble, because it makes it more difficult to drain the slough and at the same time puts that much more weight on the semifluid mass to squash it out; squashing the sloughed mass out invariably pulls with it some of the standing sections. He does this because he does not understand what produces the slough.

He does not know it is caused by too free leakage through the embankment, and insufficient land-side drainage.

"It is a well-known fact that if any vessel encountering hydrostatic pressure be leaking, the leak can be more expeditiously stopped from the outer or pressure side than from the inner side, provided, of course, the opportunities for reaching both are equally good. If a barge, for example, loaded with coal starts a leak in a side seam, it is impracticable to dig away the coal to get at it. It is equally impracticable to detect the leak by feeling along the outer side of the barge, as the inflow is too light to be detected by hand. But the deckhand can take a supply of coarse sawdust, and, by means of a long handle, can lower a cup of sawdust into the water in close proximity to the leak, shaking it gently next to the barge, and as the sawdust floats out of the cup some of it is drawn into the crack and becomes lodged; it very shortly swells and the leak is choked. Just so it is with seeping levees, but their treatment consists in dumping loose earth in the water over the river slope. The particles of earth are drawn into the interstices, become swollen, and shortly the leak is choked. The layman who sees this work in progress will immediately classify dumping loose earth in the water as sheer nonsense and not possible of practical results, but as a matter of fact, in a short while after work has been commenced, the good effects will be shown by reduced seepage, and a little later by its entire stoppage, followed by no further inclination of the mass to move, but rather by a commencing to dry out. Of course this treatment does not apply to crayfish or other leaks where there is a channel of any considerable size. If sloughs be treated when they first occur, or, better still, before they occur, when their approaching occurrence is clearly indicated to the experienced eye by the accumulation of moisture on the land slope, much annoyance and expense can be avoided."

(4) Wave wash has been mentioned as one of the causes of levee failure. It attacks the river face of levees, particularly new ones, owing to the wind or the passage of boats. When trees intervene between the levee and the river its effect is not felt, or if it is, it is not usually disastrous.

Where winds continue with considerable strength for several days, as is often the case during the spring, it is no easy task to maintain the levees. The waves roll at considerable heights and strike with force, and it requires good material to withstand their action. The following methods have been tried for the prevention of wave wash: (1) Placing a continuous strip of bagging or burlaps along the zone of the levee slope affected by the waves and securing it by pegging it to the ground; (2) placing sheets of corrugated iron, overlapping longitudinally; (3) building a bulkhead of boards along the front slope; (4) securing a floating boom, composed of logs chained together, along the levee.

The most general method used, however, is to protect the part of the slope affected with bags filled with earth, placed something after the manner of shingling a roof. Each of these methods is very expensive and has its objectionable features, and where

the extent of the damage is not likely to prove a menace to the actual safety of the levee it would probably be better to make no expenditures for protection against wave wash, as it might be cheaper to replace the material washed away after the water has subsided.

The work of restoring the slope and resodding should be done as soon after the flood as practicable, in order to allow the new material all the time possible to settle and the new sod to have all the advantage of the early season to get well set and obtain a good growth before another flood. Much good could be done by cultivating a growth of willows and cottonwood along such exposed fronts to break the force of the waves. Both of these plants are very hardy and have a rapid growth, and it is probable that plowing a few furrows and covering up live portions of willows and cottonwood would be sufficient to build up a living breakwater which would be very effective and inexpensive in future levee protection.

(5) Cutting is the fifth and last cause given for the failure of levees. This is rare, although through maliciousness, deranged mind, or other reasons levees have sometimes been cut. It is virtually impossible to guard against it unless the systems are placed under strict police surveillance, and this is not practicable.

Regarding levee failure an authority states that: "In spite of all precautions a crevasse will sometimes occur. It is generally because of some hidden defect which could not be suspected, or some circumstance that could not be seen or controlled. Levees very seldom break from sloughing, for that is generally remedied. They seldom, of late years, break from insufficient cross-section; they never break from sliding or overturning, as they ought to do by rule. They do not often break from storms, though sometimes they have had very narrow escapes. Crevasses are due mostly to three causes: to being overtopped by the water, to holes through the levees, and to weakness of the foundation. In former years the most common of all causes of breaks was insufficiency of height, because even a thin stream of water running over a bank of light or ordinary material will effect its destruction. Experience having taught that this was the chief danger to be apprehended, construction has been generally turned in the direction of increased height.

"The closing of a levee break, or crevasse, as it is generally called, is an operation which ranges, in point of difficulty, from the easy to the impossible, depending upon the height of the levee, its material, the foundation, and a number of things. The only plan which has been successful is one of the oldest employed. First, a number of bents or trestles are set up, and connected by stringers; then hand-piles are driven in front of the stringers, supported by the latter. The work is completed by filling in with sacks and earth.

"Another method consists of throwing out spur-dikes at right angles to the break and thus destroying the effect of the current, while yet another surrounds the break with a pile-dike, which is then rapidly filled with sacks of earth. As may be imagined,

earth is scarce at times of floods, and this fact will often defeat the closing of a crevasse which might otherwise be successfully stopped." * (See also p. 274.)

**Drainage of Inner Basin.**—The drainage problem of the areas inclosed along the Mississippi is a much easier one than in many other sections where levees are employed. The continuous slope of the flood-plane and its magnitude render great service in this regard. Each system of levees incloses an area which is drained by rivers, creeks, branches, etc., reaching every portion of the basin and finally emptying into the main river itself. The highest land to be found is on the bank of the river, from which point it gradually slopes back to the drainage tributary, which is generally near the hill ground. Thus it will be seen that there is a complete system of drainage for each of the various levee basins through the natural stream traversing the inclosed areas, the final outlets of which are not closed by levees because, if this were done, it would of course be necessary to exhaust the water from the basin by pumping. On account of its great quantity this would be impracticable, to say nothing of the expense, which would not be justified by the value of the lands inclosed.

By starting at the hills at the head of a basin and carrying the levee along its front as far as the outlet of the drainage tributary at the foot of the basin, it will be seen that water in entering must either pass through or under the levee, or come in through the tributary stream. That which passes through and under the embankment, while of serious import to the stability and permanence of the levee itself, may be carried into the nearest branch of the drainage system without difficulty, because its quantity at any one point is not great; but that which enters through the gap formed by the outfall may be considerable, and if it were not for the slope of the flood-plane already mentioned, the entire basin would be inundated. To more fully illustrate this we will quote Mr. Starling: † " If the front of a basin be sealed by levees, joined to the hills at its head and extending as far as the mouth of its drainage stream at its foot, the water can get access to it only through the gap caused by the entrance of the tributary. The plane of the Yazoo basin has a mean slope of about 8 inches to the mile. At a distance of 15 miles above the lower end of the levee system, therefore, the level of the back-water from the Mississippi will be about 10 feet below that of the river-water. Now, the hills which bound the basins usually approach the river gradually, so that the lower end of the basin has much less than the average width. In the instance cited the levees extended to the mouth of the Yazoo River. The area of the alluvial tract from its mouth to a line 15 miles above it would be about 250 square miles. In the case of the St. Francis it would hardly be more than half of this amount. Of these areas large tracts have so high a situation that they are several feet above back-water. Part of the remainder is irreclaimable swamp." (See Fig. 124*b*, p. 261.)

The question of drainage is, by reason of physical features, not so readily solved in many cases as in that of the Mississippi, and recourse must be had to pumping and

* "Levees of the Mississippi," p. 14.

† "Levees of the Mississippi," p. 1.

other devices. The operation is a difficult and expensive one, and involves the creation of ditches and drains in the lowest levels, with works for the regulation of flow at their outer or lower extremities. These works must not only provide for outward flow, but must prevent, as far as practicable, that which would come inward. During periods of low water these regulating gates remain open, but upon the approach of a flood they have to be closed, and the waters within the levee must remain and gradually rise over the lands unless removed by pumping. Whether this can be done economically depends upon a number of conditions, among which is whether the value of property, crops, etc., which may be injured by reason of this elevation of the water-surface, will justify the expense necessary to preserve it.

**Influence on Flood Heights.**—As levee systems are developed and narrow the field of inundation, it becomes necessary to raise their heights, because the elevation of high water is increased by reason of this restriction. The Mississippi systems are not yet complete, but sufficient progress has been made to indicate beyond all doubt that the flood level has been raised materially, and that the completion of the various systems will be the signal for greater flood levels. It will then be a struggle to hold the river within certain prescribed limits.

Experience in other countries has shown that in this struggle to keep rivers within restricted bounds they often regain their domains, and it becomes necessary to rebuild, strengthen, and increase heights of levees.

Each extension made is the signal for a further rise of elevation. An instance in point is given by De Mas: * "From the commencement of the eighteenth century the maximum heights given by a gauge near Ferrara have followed an upward march. Before 1729 the highest figures remained under 23 feet; from 1729 to 1809, or during a period of eighty years, the 23-foot mark was often exceeded without the 26-foot mark being reached. Since 1810, that is, during a period equal to the preceding, the 26-foot mark was passed five times."

Many years ago a French engineer stated that the high-water level of the Po had increased 6½ feet in two centuries, and that, while the number of breaks in the dikes was only 41 in the eighteenth century it had been 119 in the first seventy-two years of the nineteenth, 36 of which were in the year of 1872 alone.

On the Theiss, the embankment works of which are not less in importance or in results than those of the Po, there has been a steadily increasing elevation in the heights of floods. These increased levels have not come without bringing disaster. In 1879 the city of Szegedin, one of the most important of Hungary, and having a population of 75,000, was almost destroyed by inundation. The levees were repaired, rebuilt, and increased in height so that even higher floods have since passed in safety. In alluding to this the authority just quoted asks: "Will it always be thus? Will not the height of floods continue to increase in a manner dangerous even to dikes thus strength-

* "Rivières à courant libre," Art. 178.

ened? Will not the old river-beds, kept open for the escape of great floods at the location of each of the cut-offs, silt up little by little? Can the surveillance never be relaxed which was made necessary by a disaster still fresh in the minds of all? When we see what happens elsewhere we cannot fail to regard this matter with apprehension, and in rendering due credit to this remarkable work we ought not to overlook the possibilities of the future."

A similar experience is recorded with the levees of the Loire. The crown, which was originally placed 15 feet above low water, was raised to 21 feet after the flood of 1706, and even this has been found to be too low, for all the great floods have continued to rise and to surmount the embankments. After the extreme high water of 1846, an additional height of over 3 feet was placed on the levees, but the floods of 1856 and 1866 demonstrated that this raising was again insufficient.

The argument is frequently made, and is almost universally believed, that by confining a river at flood between levees increased scour of the bed and banks will take place and thus give a greater section for discharge and a greater velocity, the result of which will be to reduce not only the elevation of low water, but also that of floods.* It is quite probable that a reduction of the height of low water will follow the establishment of levees, but this does not necessarily indicate that the high-water line will also be lowered; in fact, on the Mississippi each successive exclusion of territory has been followed by a higher flood-line. In referring to the flood of 1897 Mr. Starling says:

"Its principal interest to the engineer is due to the experience which has been derived from the wholesale closure of unleveed tracts and the extraordinary elevation of its high-water line consequent thereon.

"There are two of the great basins into which the Mississippi valley is divided which have only recently been protected to any extent by levees. These are the St. Francis and the White River basins.† The former was closed during the last three years, or since the flood of 1893, to a distance, measured along the river, of about 120 miles. There still remains a gap of about 100 miles. The White River basin has been undergoing a gradual process of closure for several years. In 1893 there was a gap of about 15 miles, extending between points 330 and 360 miles, respectively, by river, below Cairo. In 1896 this gap was closed and the line of levee was made continuous from the hills at Helena to a point 8 miles above the mouth of White River.

"It is to the building of these lines and to the maintenance of the lines previously existing until a late period of the flood that the unparalleled stages attained by the water have been due."

**Efficiency.**—The engineer Belgrand has voiced his experience in regard to levees as follows: "In my opinion it is plain that even in a country where levees have existed for twenty centuries, where property has been exposed to all the consequences—I refer

* See p. 273. † See Pl. 1*a*.

to the valley of the Po—it has not been clearly demonstrated that the advantages are greater than the inconveniences."

Whether this conclusion will eventually be adopted as to the Mississippi levees time alone can decide. Without doubt there are serious objections to them, and it is probable these objections will not decrease in number or strength after the virtual completion of the systems. To quote from one of the authors just mentioned: "The best study they (levee engineers) can give to the subject—and some of them have given a great deal—leads them to think that levees are the most efficient, cheapest, and most certain means of securing the lowlands from inundation, while preserving existing conditions and improving rather than deteriorating the channel. Very considerable progress has been made toward the completion of a levee system, and it may be said that the end is plainly in sight. The engineers, then, and others charged with the responsibility of protecting the lowlands deprecate the dissipation of the none-too-plentiful funds in experiments which will certainly be costly, and which they believe will be unsatisfactory or actively harmful, when a plain road to safety lies before them. Half of this road has been traveled. It is not short at best; but it may be made indefinitely longer if we stray into every by-path that presents itself.

"The objections which have usually been urged against levees are: first, that they are too precarious—that they cannot be made strong enough to be secure against breaking; second, that they cost too much; and third, that they raise the bed of the river by confining within the channel the silt which otherwise would be carried over the banks and be deposited on the adjacent bottom lands.

"The first objection will not be entertained by any engineer when he hears that the highest banks do not exceed 40 feet, and that this height is only for a few hundred feet at most. It is, then, practicable to make the few high levees of any dimensions that may be necessary for safety with slopes of 1 to 10—if less will not do—without inordinate expense. No such proportions have ever been found necessary. About 1 to 5 is the flattest slope that has ever been used. It is not the great levees that break.

"Even when ultimate grade is attained, say 3 feet above the 'potential high water' of 1897, the average height of the levees will not exceed 18 feet, of which 3 feet will be a margin against storms or accidental deficiencies, settling, etc., leaving a water-head of 15 feet. Extreme high water will last only a few days; within a foot of extreme height it may last three or four weeks. For such embankments, in ordinary soils, slopes of 1 to 3, with banquettes, will be sufficient. . . . As to cost, it may be said that levees are the least expensive means of reclaiming overflowed lands that have ever been proposed. . . . The idea that confining a river should cause it to deposit silt is so contrary to reason that it is a wonder it ever obtained credence at all. Transporting power is generally believed to be proportional to the square of the velocity. The confinement of the stream unquestionably gives increased current. Undoubtedly, in retaining the

water within the channel we also retain the silt, but at the same time we retain the vehicle by which to carry it, and give the vehicle greater capacity.

"General Comstock's conclusion was that in the cases of the Po and the Rhine the rise of bed, if any, was insignificant, and that in the case of the Mississippi there is no evidence of any at all."

While sufficient time has not elapsed to detect any decided change in this respect resulting from the extension of the levee system on this river, some instructive facts have been developed. Between 1881 and 1883 surveys were made with great exactness and detail for a distance of 1063 miles, reaching from Cairo to the mouth. An average of four cross-sections was taken to each mile, and permanent gauges were established. Between 1894 and 1896 the 425 miles below the mouth of the Arkansas River, com-

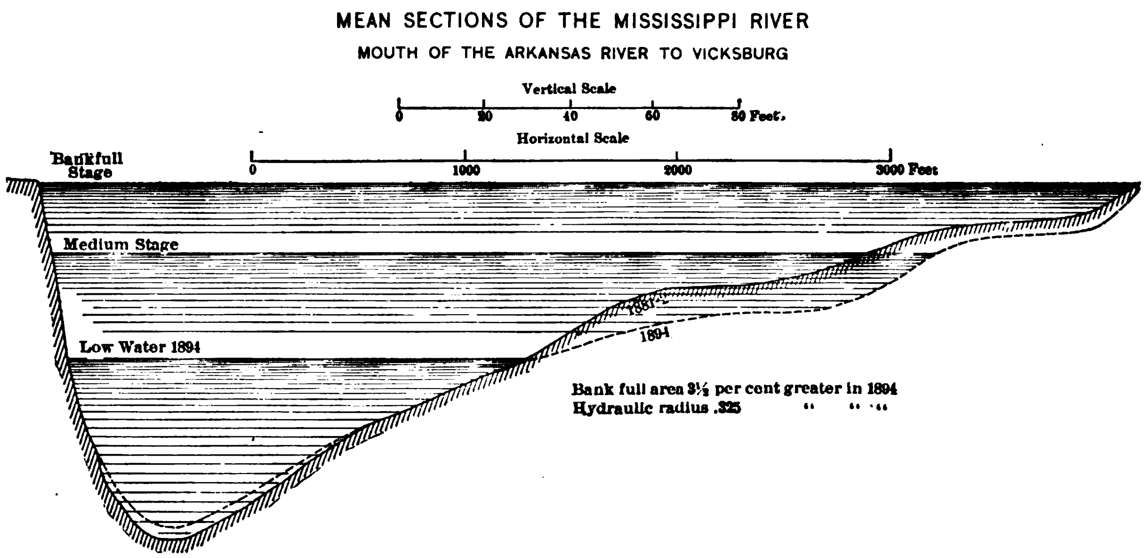



FIG. 125.

prising the portion which had been most improved by levees in the interval, were resurveyed with similar care and detail, and the corresponding cross-sections platted and calculated. The comparison of the two periods, which it is reasonable to believe would indicate the general tendencies towards change in the regimen of the stream during the thirteen years, showed that the river was evolving a channel more uniform and of greater capacity, the mean of the sections being shown in Fig. 125. It was found also, from these and other data, that the crests of all bars had become lower, and that with equal flow and channel depths the low-water surfaces had become reduced in height from 1 to 2 feet.*

Comparative surveys of the Atchafalaya River, an important side-outlet, about 150 miles in length, for the Mississippi during floods, have also afforded some light on

* Proceedings Am. Soc. C.E., vol. li, p. 331 and after.

this subject. At low-water the Mississippi rarely pours any water into this river, while in high floods it has delivered more than 400,000 cubic feet per second. The head of the stream was obstructed up to about 1860 by an immense body of drift nearly 20 miles in length, consisting of logs, trees, and débris of all description, called in America a " raft." This was finally cut out, and the increased flow in the stream resulted in a considerable increase in erosion and flood heights. The first survey was made in 1880, and showed unusual variations of section; the stream in many places was deepest where it was widest, shallowest where contracted, and larger at the head than at the mouth. These variations were supposed to be due to the disturbances of flow caused by the raft. Between 1880 and 1904, in which last-named year a second survey was made, levees were gradually built along the banks, so that the flood flow through the most irregular portions was brought more or less under control. The last survey showed that there had been a definite tendency towards the creation of a more regular bed; the river had widened and deepened its bed where the floods formerly spread over the banks; pools had filled up where the cross-section had before been too large; and an examination of the upper 14 miles, the earliest part improved, showed that the final tendency was towards a regular bed of about 56,000 square feet. Individual sections taken lower down, where the river was still at work, showed changes amounting to double the width and to an increase in depth of 80 feet. The disturbances in the current during floods, especially where the river emerged from the confined channel between the levees and entered its unimproved bed, were tremendous.*

**Seepage from Levees and Effect on Crops.**—A continued flood will sometimes affect crops near the levees by the raising of the ground water and the leakage through the banks. Experience along the Mississippi, however, has shown that damage from this cause rarely occurs, partly because the floods usually come during the winter or early spring before the land is cultivated, and partly because the seepage does not often extend further back than a few hundred feet, unless the flood is of long duration. Where the drainage is poor, the effects have been observed up to distances of 3000 feet, but under usual conditions good ditching will limit the extent to much less. On account of such seepage the land just behind the levee is generally planted with grass for pasture, as on any other crop, such as cotton, the effect is fatal.

**Closing a Crevasse.**—The work of closing a crevasse, or break in a levee during a flood, is usually attended with considerable difficulty, but unless promptly checked the break will quickly increase in width and depth and result in serious damage. The water not only cuts back the broken ends of the levee, but also attacks the ground and erodes it, tearing great holes, some of which have covered several acres and reached depths of 50 feet. The crevasse widens rapidly at first, but as the flow increases and the country behind begins to fill up with water, the current slackens. In its first stages it is often strong enough to uproot trees and destroy buildings, and may ruin much land by cover-

* Transactions Am. Soc. C.E., 1906, J. A. Ockerson.

ing it with sand.* Crevasses on the Mississippi have sometimes reached widths of more than 10,000 feet.

These breaks, if taken in time, can sometimes be checked by throwing in brush and sand-bags, or by building at once a new levee of sand-bags on the river side. By the term "sand-bags" is meant sacks filled with earth or sand; on the River Po sacks are used filled with the gravel composing the alluvial plain, as it can generally be obtained as easily as earth. If the water has gained much headway, however, the closing becomes almost impossible until the river has fallen or the flow becomes slack. For this reason the Mississippi engineers rarely attempt to check or close a crevasse which has gained headway until the country behind has filled with water sufficiently to check the current, at which period the exposed ends of the levees can be revetted with sand bags and final means taken to close the break. The following method, used in closing a crevasse on the Mississippi during the flood of 1903, is typical of the means employed. As soon as material could be secured, together with the necessary towboats and barges, work was commenced by driving piles at the levee and about 200 feet from its caving ends. The piles were of small size, so as to permit easy handling, placing, and driving, most of them being of sawed timber 4 inches square. They were driven from 3 to 4 feet centers, and with 6 to 8 feet penetration, and covered a width of about 60 feet or approximately the base-width of the levee. As the work was carried out into the current each pile was braced to the last ones and a working platform was laid on top, and sand-bags were dropped between them to stop further erosion of the ground. When all were driven, the lines extended in a flat arc of a circle enclosing the break on its river side. Other piles were then put in to check further the rush of water, and other sand-bags with armfuls of grass and hay were dropped in until they reached the surface and cut off the flow, forming a new levee of about the same section as the old one. The piles were driven by gangs of men hammering them with heavy timbers. About two weeks were occupied in closing the crevasse, and although the expense was some $40,000, it was estimated to have saved crops which had a value of more than twenty times that amount. When the flood-season had passed, a new levee was built encircling the break, and forming once more a continuous protection, in a manner similar to that shown by Fig. 126. A double row of sheet-piling filled with earth between, like a cofferdam, is sometimes driven outside the cribbing, as an additional protection. (Fig. 126*a*.)

Along the Nile, which is enclosed by levees from Assuan to the sea, a distance of more than 600 miles, unusual watchfulness is employed in time of flood to guard against disaster. In Middle Egypt a very high rise attains to about 7 feet above the surrounding country, while at the Delta it may range from 7 to 12 feet above. The country is so thickly settled that any break in the levees might lead to a serious loss of life. When danger threatens two watchmen are stationed day and night all along the banks at

* The method of dumping riprap from barges until the break was closed has been used in France. For a description of such methods see "Engineering News," August 1, 1912.

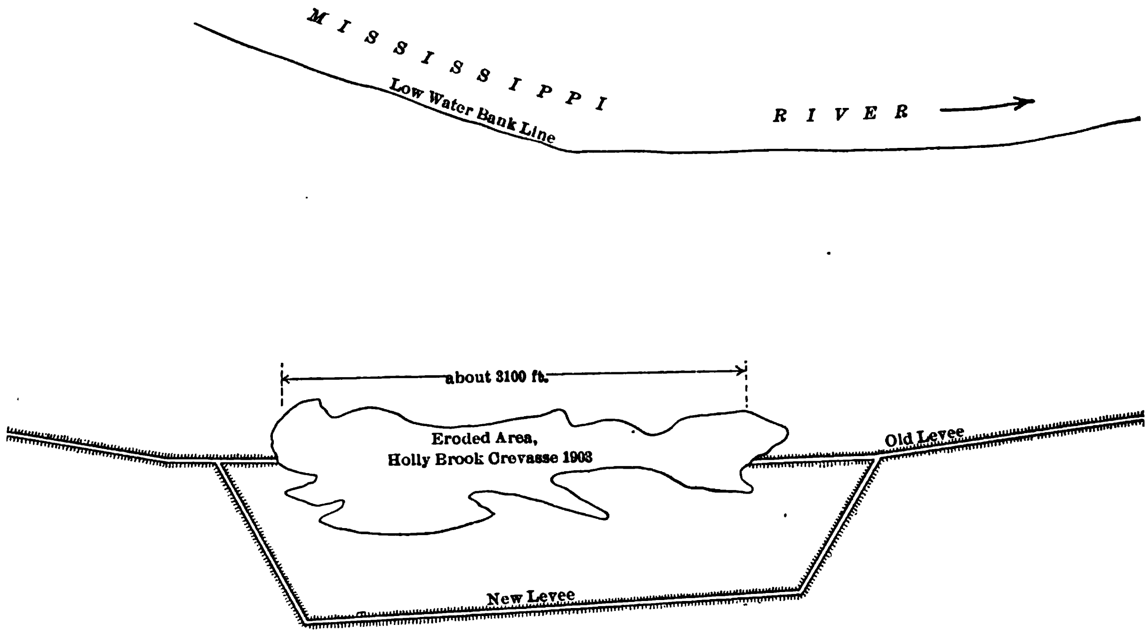
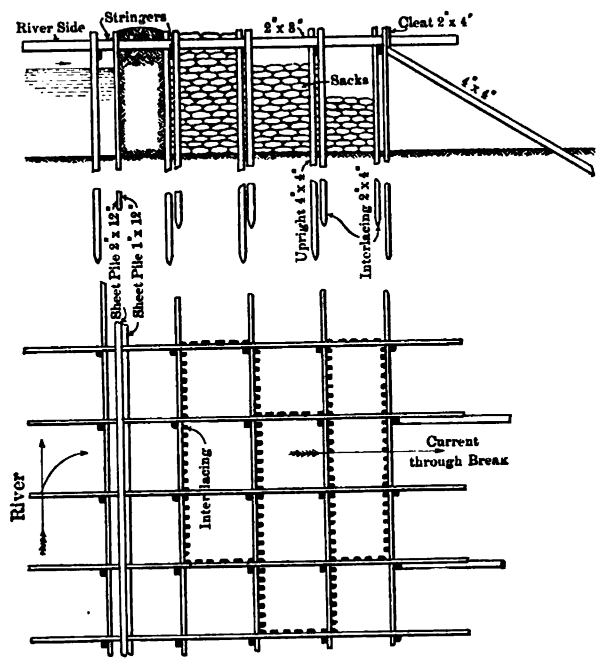



FIG. 126.—Erosion and New Levees, Holly Brook Crevasse.




PLAN AND SECTION OF CRIBBING
USED FOR CLOSING A CREVASSE

FIG. 126a.

distances of about 150 feet, and gangs of 50 to 100 men are kept in reserve at every point of danger. Steamers and boats carrying sacks, stone and stakes are within reach, and the face of the levees for nearly the entire length of the river is protected by stalks of cotton and corn fastened to stakes, to prevent the waves from washing the banks. Prior to the British occupation additional measures of precaution were taken by the authorities. Thus in 1878, when a dangerous break occurred at Mit Badr, orders were telegraphed to throw the engineer and the superintendent of the division into the river. Twelve hours' grace was given them by the local Governor, however, in which to close the crevasse; this they did and were in consequence afterwards pardoned, but the superintendent's hair turned white during the twelve-hours interval.*

**Economic Results of the Mississippi Levees.**—The economic possibilities of the levee system of the Mississippi are illustrated by the development of the Yazoo Basin, which was formerly subject to inundation by floods. The Yazoo River is a secondary affluent flowing along the foot of the bluffs which form the eastern boundary of the main alluvial valley, and it drains the bottom lands between Memphis and Vicksburg. In 1880, four years prior to the commencement of effective levee protection in it, this territory, comprising only about one-third of the alluvial valley of the Mississippi River, but with an area of some 6500 square miles or more than 4,100,000 acres, was sparsely settled and practically without railroad communication. Three-fourths of it was still virgin forest, and many places were covered with water during high floods to a depth of 25 feet. Twenty years later there was a network of 800 miles of railway penetrating the district; the population had grown enormously, and flourishing crops were raised annually on lands which had in past times been regarded as irreclaimable swamps. The effect on sociological conditions was equally marked—churches, schools, and settlements had multiplied and the inhabitants were better housed, fed and clothed.† The alluvial lands of the Mississippi valley are among the richest in the world, and when stripped of timber they become valuable for agriculture, contrasting in this respect with many districts of higher level whose value consists almost entirely in their forest growth. The change of conditions above described existed at the same period (1900) to a greater or less extent throughout all portions under the protection of levees, and while crevasses were of occasional occurrence, the resulting damage was local, and unworthy of comparison with the benefits which had accrued to the valley at large.

It is estimated that there are nearly 30,000 square miles, or about 20,000,000 acres, of fertile soil along this river below its junction with the Ohio which can be cultivated with the protection of levees.

**Foreign Levees.**—Mention has been made of the levees along the Po, the Theiss, and the Loire, and owing to the extent and age of these works further information in regard to them will be of interest.

* "The Nile in 1904," Sir Wm. Willcocks.
† Report of the Mississippi River Commission, 1904.

*River Po.*—Usually the levees on the Po are located at a considerable distance from the banks, but occasionally they come quite close, and at such places are protected by revetments or fascines, an inner levee being frequently built behind them for fear of accident. They extend up each side of each tributary to such distances as floods are to be expected. In a word, the great submersible plain of the Po is guarded by a network of embankments enveloping each affluent and laying out for each a minor bed which must be followed by all the water falling in the valley. These levees are located wider apart in the neighborhood of tributaries of great flow in order to provide a greater area for the increased discharge, thus forming reservoirs in which is gathered not only the surplus water, but tremendous quantities of sediment as well. In regard to the result of this method De Mas says: * "The controlling levees have been spaced so that in the localities where affluents abound the major bed forms a sort of reservoir, in which are stored not only the floods, but also the deposits carried by turbid waters. It follows from this arrangement that the major beds of the Po and of its affluents in their lower parts serve as regulators to each flood descending from the mountains; and after having distributed it they lead it to the sea through a contracted outlet which modifies the velocity and the disastrous effects caused by the raising of the flood-level above Panaro. This distribution was first brought to attention when Lombardini stated two remarkable facts which are shown also in the report of the engineer Baumgarten. The first is that the discharge of a flood is very nearly the same at Tessin, at Cremona, and in the vicinity of Ferrara. The second is that while all its affluents together discharge 528,000 cubic feet per second, the discharge of the Po is about 176,000 feet for the same unit of time. The greatest part of this remarkable result should be attributed to natural circumstances. It is certain that the affluents from the Apennines discharge before those from the Alps; and there is no doubt that the lakes Maggiore, Como, Garda, and others retard the most copious torrents. Thus it was found that at Lake Como the maximum discharge in the flood of September, 1829, was at the entrance 68,500 cubic feet per second, and only 28,400 cubic feet at the exit. It will be seen, therefore, that the flood was diminished in passing through the lake in the ratio of 2.4 to 1. But it is equally certain that in the establishment of levees whose effects are to produce certain contrary results natural laws must not be neglected, and a knowledge of these must govern us accordingly. Thus, according to the same experienced engineer, the volume stored between the levees of the Po and its affluents, from Casale to the sea, is 66,739,000,000 cubic feet, which corresponds to more than four days' discharge of the river at the rate of 181,300 cubic feet per second. In reality the hand of man has created a vast regulating reservoir which affects the regimen of the river in the same way that the lakes cited above affect its affluents."

The principal territory under protection lies between the tributaries Ticino and

* "Rivières à courant libre."

Panaro, and comprises 1,730,000 acres. The length of the embankments is 322 miles. Sections are shown on Fig. 124*a*, p. 254.

*The Theiss.**—The Theiss flows through the plain of Hungary, which in general consists of a layer of vegetable earth overlying a thick stratum of compact black clay. The former consititutes the banks and the latter the bed of the stream. Like most rivers the slope is great in the mountainous portion, diminishing continuously in the plain. The heights of the floods increase as the slope decreases. They come in the spring, and are caused by the melting of snow in the Carpathian Mountains. They are slow, often taking several weeks in which to reach their full height, and their duration at the highest level is generally several days. The maximum discharge at Szegedin is given as 123,200 cubic feet per second. The river always maintains a good navigable depth.

Levees continue without interruption along the Theiss and its principal tributaries. (See Fig. 124, p. 252.) Their entire surface is sodded, but the sod is not always proof against floods, and it is found necessary to resort to other means of protection, such as fascines, planting of willows, etc. These embankments are usually at some distance from the river bank, generally from one-half to three-quarters of a mile where the river is 600 to 800 feet in width, and are never built within less than one-quarter of a mile of the river-bank. Although of recent construction they are quite as important as those of the Po, and the results have been equally satisfactory. Their maintenance is largely in the hands of the adjacent landowners. Their total length in 1908 was 2062 miles, and they protected 6,425,000 acres. In addition to these were other embankments along the Hungarian Danube with a length of 1552 miles, protecting 2,770,000 acres, making a total of 3615 miles, protecting 9,140,000 acres. An extensive system of drainage, involving more than 6000 miles of ditches, has also been constructed, the water being taken through the levees by culverts. The cost of all works up to 1908 was about $120,000,000, of which two-fifths was paid by the State.†

*Loire.*—The levees on the river Loire are usually built upon one side of the stream only, the opposite side being a slope or a bluff. This arrangement is occasionally varied where the river crosses from one side of the valley to the other. The discharge below Bec d'Allier is about 352,000 cubic feet per second in the greatest floods. This great quantity of water is concentrated in a narrow bed, the width of which was not fixed by a consideration of all the necessities of the case. The addition of new levees has complicated the matter, and it is stated that the crowns have been raised 9 feet within two hundred years. The total length of the embankments is about 302 miles, and the protected area comprises 235,000 acres.

*Holland.*—This country has about 3,200,000 acres protected against floods, and about

* Annales des Ponts et Chaussées, 1890; Proceedings International Congress of Navigation, 1908.

† Proceedings International Congress of Navigation, Section 1, Question 5, 1908.

210,000 acres which were formerly lake beds or expanses of sea.* Sections of the levees are shown on Fig. 123, p. 251.

*Nile.*—See p. 275.

**Cost of Mississippi Levees.**—A report of 1911 gives the following:

| | |
|---|---|
| Miles of levees required for the completed system (82% built) | 1565.25 |
| Contents of levees then built, cubic yards | 241,040,518 |
| Estimated final contents, cubic yards | 295,000,000 |
| Cost per cubic yard | 11 to 35 cts. |
| Average annual loss by crevasses, caving, etc. (maximum for one year, 4%) | 2½% |
| Average annual cost of maintenance during eight years, 1½% per annum of original cost. | |
| Approximate area protected, square miles | 26,569 |
| The total of all expenses for levees, dredging, bank protection, etc., on the Mississippi from Cairo to the Gulf up to 1911 was about | $62,230,000 |
| (equivalent to about $45,000 per mile). | |

A typical plan of a portion of the system will be found on p. 261, and a general map of part of the lower Mississippi River on Pl. 1*a*.

* Proceedings International Congress of Navigation, Section 1, Question 5, 1908.

# CHAPTER VII.

## STORAGE RESERVOIRS.

**General.**—Nature has indicated one satisfactory method of improving the navigability of watercourses, in the lakes which lie at the foot of mountainous regions and from which rivers flow. By them the length of the navigable season is increased and the danger from floods is decreased, and the lesson taught is that where artificial lakes or reservoirs can be constructed near the sources of streams, the waters falling in the various basins leading to these reservoirs may be usefully stored up. Not only will excess of water be thus held back while that entering lower down is making its escape, thus preventing a flood, but it may be drawn out as required by the necessities of navigation and to its great benefit.

About the year 1800 Thomas Telford, a distinguished civil engineer of England, wrote a work advocating the storage of flood-waters and urging its adoption for the improvement of the navigation of the river Severn. His idea was " to collect the flood-waters into reservoirs, the principal ones to be formed in the hills of Montgomeryshire, and the inferior ones in such convenient places as might be found in the dingles and along the river. By this means the impetuosity of the floods might be greatly lessened, and a sufficient quantity of water preserved to regulate the navigation in dry seasons, etc. This, it is thought, might now prove the simplest and least expensive mode of regulating navigable rivers, especially such as are immediately on the borders of hilly countries." Another English engineer, William Jessup, also gave the matter considerable thought, and expressed the opinion that " rivers may be rendered nearly uniform throughout the year by reservoirs." Mr. Rennie, however, also an English engineer of distinction, ridiculed the ideas of Telford and Jessup in regard to the correction of floods by such means.

Charles Ellet, Jr., and Elwood Morris, both well-known engineers of their day, strenuously advocated the reservoir plan for the Ohio River. In 1857, however, W. Milnor Roberts, one of the ablest authorities on river improvement this country has had, carefully investigated the plan and made the following statement: " My own careful investigation of the subject of controlling the floods of the Ohio by means of artificial reservoirs satisfied my mind conclusively that such control by any human means attainable within the practicable limits of cost is impossible." Mr. Roberts gave his views in the Journal of the Franklin Institute in 1857. He proved from an examination of the records of the floods on the upper part of the Ohio, that some of the highest floods occurred when such

reservoirs, had they been in existence, would have been full. Such being the case, he argued that they could not have materially aided in restraining those floods, and that this would certainly be the case almost every year owing to the irregularity of the periods when great floods occur. "If by possibility there could be a gigantic dam 400 feet high at Wheeling, sufficient actually to stop and absolutely to control all the water of the 27,337 square miles of drainage above Wheeling, it could not restrain any portion of the flow from the remaining 189,663 square miles of the Ohio valley, nearly seven times the area. We should even then have control of only about one-ninth of the Ohio River territory. In my opinion, the scheme of controlling or equalizing the floods of the Ohio River by means of artificial reservoirs is impracticable." *

This reasoning, which is based on the great cost of the works, is applicable to many other cases as well as to that of the Ohio.

After the inundations which devastated France in 1846, 1856, and 1866, the question of reservoirs was widely discussed, as mentioned farther on, but their excessive cost prevented their application on a great scale, and a French authority has in recent years stated that "the idea of modifying immediately the régime of inundations by the creation of a system of reservoirs is now considered as unrealizable." Within recent years, however, the question of first cost has been modified by the possibilities of power development.

The question of storage reservoirs has been exhaustively entered into by a United States' Government report, from which the following are extracts: †

**Natural Reservoirs.**—"Nature presents abundant examples of the effective control of stream-flow through the agency of reservoirs. There are indeed comparatively few streams whose flow is wholly uninfluenced by such action. The most perfect example in the world, both as to the magnitude of the stream and the completeness of control, is the St. Lawrence River, embracing the great chain of North American lakes. (Fig. 127.) Considering only that portion of the system which lies above the Falls of Niagara, let the flow at the outlet be compared with that of other streams of similar magnitude. For this purpose take the Niagara River at Buffalo, the Ohio at Paducah, Ky., the Missouri at its mouth, and the Mississippi just above the mouth of the Missouri. The following table gives the area of watershed in square miles and the mean annual discharge in cubic feet per second of each.

| | Niagara. | Ohio. | Missouri. | Mississippi. |
|---|---|---|---|---|
| Watershed........ | 265,095 | 205,750 | 530,810 | 171,570 |
| Discharge......... | 232,800 | 307,000 | 100,000 | 130,000 |

* A commission appointed by the city of Pittsburgh, Pennsylvania, to look into the question of reducing floods there reported in 1911 in favor of a project for building 17 reservoirs in the drainage area above the city at an estimated cost of $20,000,000. The flood damage to the city in the ten preceding years was estimated as $12,000,000. See also Transactions Am. Soc. C. E., 1912.

† "Reservoir Sites in Wyoming and Colorado," Captain Hiram S. Chittenden, Corps of Engineers, U. S. A., House Doc. 141, 55th Congress, 2d Session, 1898.

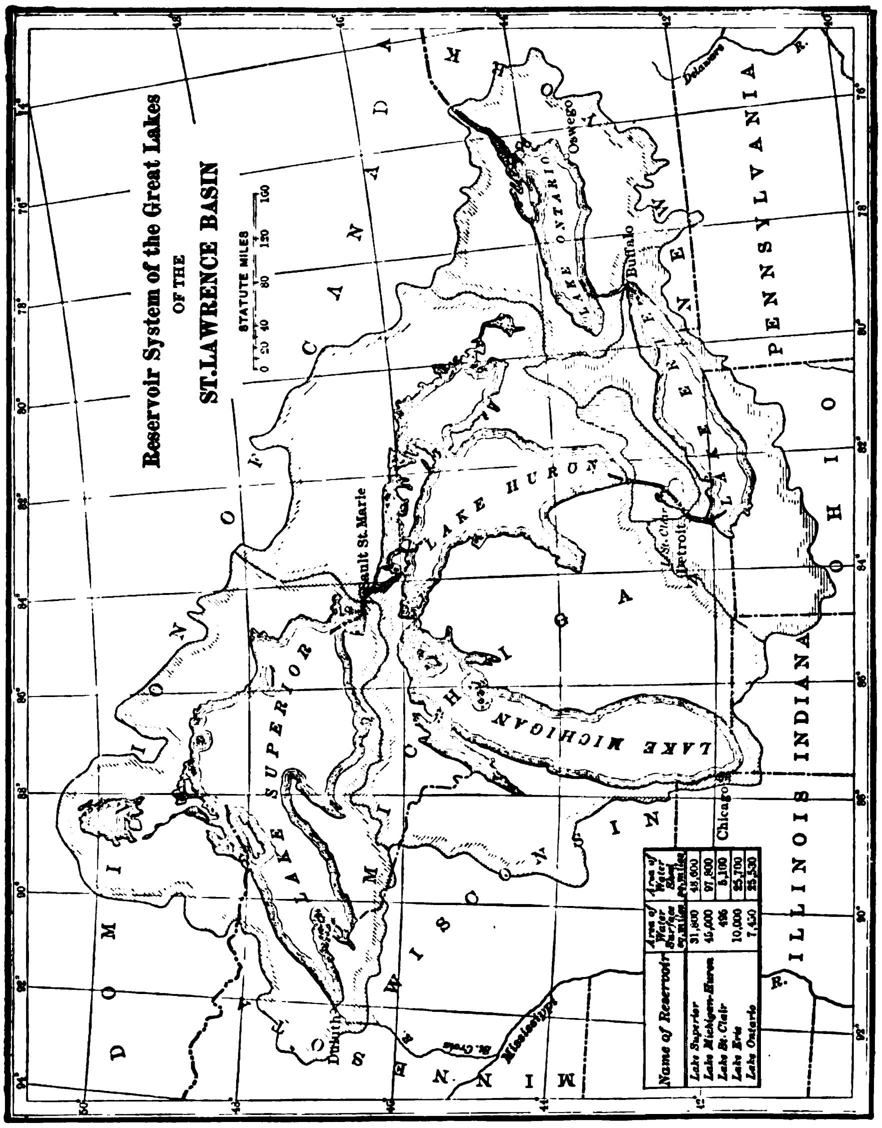

Reservoir System of the Great Lakes
OF THE
ST. LAWRENCE BASIN
STATUTE MILES
0 20 40 80 120 160
DOMINION OF CANADA
LAKE SUPERIOR
LAKE HURON
LAKE MICHIGAN
LAKE ONTARIO
Sault St. Marie
Duluth
Chicago
Detroit
St. Clair
Buffalo
Oswego
MICHIGAN
WISCONSIN
MINNESOTA
NEW YORK
PENNSYLVANIA
OHIO
INDIANA
ILLINOIS
Mississippi
Delaware R.
Name of Reservoir
Area of Water Surface sq. miles
Area of Water Shed sq. miles
Lake Superior 31,800 48,600
Lake Michigan-Huron 45,000 97,800
Lake St. Clair 495 5,100
Lake Erie 10,000 25,700
Lake Ontario 7,450 25,530

FIG. 127.

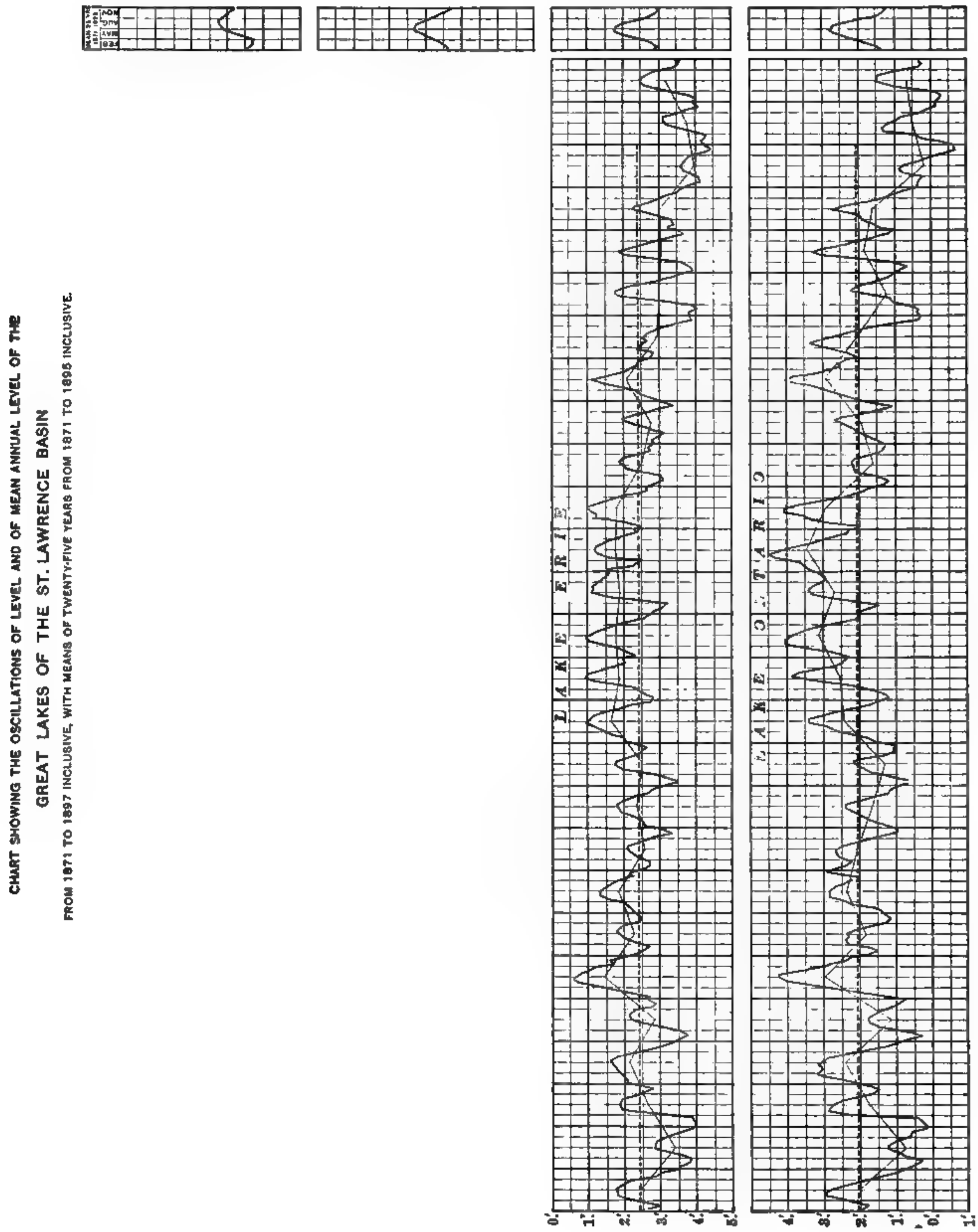


CHART SHOWING THE OSCILLATIONS OF LEVEL AND OF MEAN ANNUAL LEVEL OF THE
GREAT LAKES OF THE ST. LAWRENCE BASIN
FROM 1871 TO 1897 INCLUSIVE, WITH MEANS OF TWENTY-FIVE YEARS FROM 1871 TO 1895 INCLUSIVE.
LAKE ERIE
LAKE ONTARIO

FIG. 128.

"The above discharge for the Niagara River is based upon twenty-five years' record (1871–1895); that for the Ohio and Upper Mississippi upon six years' record (1880–1885); and that for the Missouri upon twelve years' record (1879–1890).

"The maximum and minimum discharges, except for Niagara, show a much greater divergence, the ratios of $\frac{\text{maximum discharge}}{\text{minimum discharge}}$ for 1883 being as follows:

"Niagara, 1.19; Ohio, 28.22; Missouri, 29; and Upper Mississippi, 10.29.

"This striking dissimilarity in the regimen of streams of similar magnitude, and, with one exception, of similar climatic conditions, is entirely due to the reservoir action of the Great Lakes. Of that portion of the St. Lawrence drainage-basin which lies above Niagara Falls, viz., 265,095 square miles, 87,400 square miles, or almost one-third, is made up of the water-surfaces of Lakes Superior, Michigan, Huron, and Erie. One foot upon this immense area represents 2,436,000,000,000 cubic feet—greater than the excess of the late Mississippi River flood at Cairo above the bankful stage.

"The mean annual fluctuation of Lake Superior, based upon twenty-five years' observation (1871–1895), is 0.93 foot; of lakes Michigan and Huron, 1 foot; of Lake Erie, 1.16 feet. This fluctuation represents an annual storage of 2,419,000,000,000 cubic feet of water, equivalent to about 153,000 cubic feet per second for a period of six months. The maximum annual fluctuation during the above period is just about twice the above mean, and of course represents twice as much water stored.

"In addition to the annual fluctuation, there is constantly going on a periodic change which often requires several years to complete the cycle. As an illustration of this characteristic of the Great Lakes take the period of eight years from 1872 to 1879, inclusive, during which the mean annual level of the four upper lakes rose for a period of four years and fell during the following three years. The rise in mean level was, for Lake Superior, 1.03 feet; for Lakes Michigan and Huron, 2.02 feet; and for Lake Erie, 1.97 feet. The total storage represented by this rise of mean level was 4,000,000,000,000 cubic feet. The fall in mean level following the rise was, for Lake Superior, 1.63 feet; for Lakes Michigan and Huron, 1.46 feet; and for Lake Erie, 1.17 feet—equivalent to 3,627,000,000,000 cubic feet. After this fall the mean level began to rise again. (See Fig. 128.)

"The foregoing figures convey some faint idea of the magnitude of the storage of the Great Lakes, and of the way in which it operates to preserve a balance not only between the wet and dry seasons of each year, but between those cycles of wet and dry years which are continually recurring. These reservoirs absorb the flood-waters of spring and pay them out in the following dry season, thus preventing floods on the one hand and low water on the other. And while these seasonal changes are going on the lakes respond to the varying conditions of longer periods, levying upon years of more than average precipitation in order to maintain a flow in the outlets during the years of deficiency which are certain to follow.

"The result of this storage action of the Great Lakes is to produce a river system radically different in its general characteristics from nearly all other streams. Such conditions as high and low water, as elsewhere understood, are here entirely unknown. Commerce pursues its way through these lakes and rivers without serious hindrance except when ice closes the way; and the river and harbor engineer has little to do with low-water problems or protection against floods, but rather with the deepening of harbors and connecting channels for an ever-increasing size of vessels and volume of commerce.

"The vital function which the fluctuation of levels, both annual and cyclic, plays in the economy of the Great Lakes is doubtless not generally appreciated even by the engineering profession. Only recently distinguished engineers have boldly asserted that this fluctuation of levels is an evil which must not be suffered to continue, and they have proposed plans by which it may be corrected. Yet nothing is more certain than that any curtailment of these fluctuations, either annual or cyclic, can be accomplished only by a corresponding curtailment at certain seasons of the discharge of the lake outlets.

"Besides the Great Lakes of the St. Lawrence basin there are many other natural reservoirs in various parts of the world. In order to convey some idea of their geographical distribution, magnitude, and regulating influence upon stream-flow, the following list of the more prominent examples is presented:

LIST OF PROMINENT EXAMPLES OF NATURAL RESERVOIRS

| Name of Lake. | River System. | Elevation above Sea-Level. Feet. | Area. Sq. Miles. | Percentage of Area to Entire Watershed. | Storage in Billion Cubic Feet Represented by a Fluctuation of 1 Foot. | Remarks. |
|---|---|---|---|---|---|---|
| Superior | St. Lawrence | 601.6 | 31,800 | 39.5 | 886.5 | Authority: Report of United States Deep Waterways Commission, 1896. |
| Michigan | St. Lawrence | 581.2 | 22,400 | 32.9 | 638.4 | |
| Huron | St. Lawrence | 581.2 | 23,200 | 30.8 | 646.8 | |
| St. Clair | St. Lawrence | 575.3 | 495 | 7.3 | 13.8 | |
| Erie | St. Lawrence | 572.8 | 10,000 | 34.5 | 278.3 | |
| Ontario | St. Lawrence | 246.3 | 7,450 | 22.6 | 207.7 | |
| Baikal | Yenisei | 1,360.0 | 12,430 | 6.0 | 346.5 | Encyclopædia Britannica. Watershed scaled from map. |
| Victoria Nyanza | Nile | 4,000.0 | 27,000 | 24.0 | 752.7 | Encyclopædia Britannica. Watershed scaled from map. Accuracy only approximate, as data are of doubtful authenticity. |
| Albert Nyanza | Nile | 2,300.0 | 2,000 | 12.5 | 55.8 | |
| Tanganyika | Congo | 2,700.0 | 12,650 | 12.0 | 352.7 | |
| Nyassa | Zambesi | 1,600.0 | 9,000 | 24.0 | 250.9 | |
| Titicaca | Desaguadero | 12,600.0 | 3,200 | 17.0 | 89.0 | Encyclopædia Britannica. Watershed scaled from map. |
| Geneva | Rhone | 1,218.0 | 223 | 8.0 | 6.2 | |
| Constance | Rhine | 1,306.0 | 208 | 4.0 | 5.8 | |
| Neuchatel | Rhine | 1,427.0 | 92 | 11.9 | 2.6 | |
| Como | Po | 670.0 | 64 | 4.0 | 1.8 | |
| Maggiore | Po | 646.0 | 83 | 3.3 | 2.3 | |
| Garda | Po | 320.0 | 135 | 19.4 | 3.8 | |
| Yellowstone | Missouri | 7,741.0 | 139 | 15.9 | 3.9 | United States Government reports. |

"The moderating influence of any of these lakes upon the streams below them is, of course, very great. Lake Geneva, for example, in the great flood of 1856 discharged

only 11,400 cubic feet per second at the maximum, as against 56,480 cubic feet which it was receiving from its watershed.

"In Italy the lakes on several of the northern tributaries of the Po have long been noted for the control which they exercise over the streams flowing through them. The violent and destructive floods which are characteristic of other tributaries of the Po are largely absent from those streams which flow through the lakes.

"The flow of the Rhine in its upper source is said to be subject to much less variation than other streams similarly conditioned except as to natural reservoirs.

"There are many thousands of other lakes scattered over the globe that act as regulators of the streams which drain them, their efficiency in this respect being proportional to the percentage which their areas bear to the tributary watersheds. Certain it is that the aggregate influence of these reservoirs is very great, and the striking difference often noted in the characteristics of the flow of streams with similar watersheds may largely be traced to this cause.

**Artificial Reservoirs.**—"While it is impracticable to imitate nature on the scale of her own work in the construction of reservoirs, her example has nevertheless been followed very extensively on a smaller scale. In fact, works of this character have been built for a variety of purposes since the remotest antiquity. The storage of water for feeding canals is a prominent example. The greatest reservoir systems yet constructed have been designed to maintain the navigable condition of natural waterways. Many reservoirs have had as a prominent reason for their construction the prevention of floods in the valleys below them, although this has seldom if ever been an exclusive reason. Storage of water for city supply, the development of power, and other industrial uses, is one of the most familiar of modern enterprises. Finally the field of irrigation, which already presents many examples of great reservoirs, bids fair to outstrip all other fields in the production of works of this character. In all these examples of reservoir construction the purpose has been to correct the inequalities of nature—to prevent the rapid and destructive flow of rivers at seasons when not needed, and to augment and re-enforce that flow when the need does exist.

"One of the most extensive artificial systems ever built is to be found in Russia at the headwaters of the Volga and Msta rivers. The Volga River, the greatest in Europe, 2425 miles long, and navigable nearly its whole length, rises in the province of Tver, within 200 miles of St. Petersburg, and empties into the Caspian Sea in the opposite extremity of European Russia. The Msta River has its scources interlaced with those of the Volga, but flows in the opposite direction, and its waters find their way, through the Volkhoff River, to Lake Ladoga, and ultimately to the Baltic Sea.

"The sources of the Volga and Msta are in a flat, marshy, wooded country, about 665 feet above sea-level, covered with innumerable lakes, presenting conditions not unlike those which prevail at the sources of the Mississippi River in our own country. For a long period in the past these two river systems were connected by artificial

waterways, and the seaport of the upper Volga was upon the Baltic. The extreme low water which is characteristic of the Volga and other Russian streams prevents navigation in their natural condition except in seasons of high water. To ameliorate this condition, advantage was early taken to the exceptional reservoir facilities offered by the lakes referred to, and dams of a cheap character were constructed across their outlets. The reservoir system has now been developed to great perfection and effects an important improvement both in the Volga and the Msta, rendering them navigable for nearly three months longer than they would be without this aid.

"These reservoirs store about 35,000,000,000 cubic feet of water in all, of which 20,000,000,000 can be used in the Volga and 20,000,000,000 can be turned in the other direction, there being apparently a storage of about five or six billions that can be used in either direction. The largest and most important of these reservoirs, and one of the largest in the world in point of capacity, although insignificant in depth and containing-dam, is the Verkhnevoljsky reservoir. So slight is the fall of the stream in this region that, although the dam produces a maximum elevation of water-surface at its site of only about 17.5 feet, the water backs up a distance of about 60 miles and includes several lakes. The low-water season capacity of this reservoir is about 14,000,000,000 cubic feet, and the average season storage is much greater. Its effect upon the low-water flow of the river below the dam is to raise its normal surface 2.8 feet at Rjef, 96 miles below; 1.4 feet at Tver, the mouth of the Tvertsa, 212 miles below; and 0.14 foot at 410 miles below. At the mouth of the Tvertsa the storage of the Zavodsky reservoir comes in and helps out the navigation below. The total navigable distance on the Volga over which the beneficial influence of these reservoirs is felt is upward of 450 miles.

"On the Msta slope there are no fewer than ten important reservoirs, all of them being on the sites of natural lakes, the total storage aggregating about 14,000,000,000 cubic feet. As already stated, about 6,000,000,000 cubic feet of storage which really lies on the Volga slope, including the Zavodsky reservoir, formerly was and still can be turned into the Baltic drainage. This entire system of summit reservoirs that can be used to feed the Msta is called the Vychnevolotsky system. It affords material improvement to the navigable condition of Msta and Volkhoff rivers during the period of low water.

"The system of reservoirs just described is certainly a great success, and upon it much of the prosperity of the surrounding country depends. It is probably the most complete example in the world of the joint results of flood prevention and the improvement of navigation produced by artificial reservoirs. It has an importance, however, which it could not have in this country, even with equal physical advantages, for railroads here do a far greater proportion of the transportation business than in Russia. But the example shows how far favorable natural conditions can be made to improve the low-water conditions of streams.

"The largest artificial-reservoir system ever yet constructed is that at the head-

waters of the Mississippi River. The natural conditions prevailing in that region are very similar to those in Russia just described. The country about the sources of the Mississippi, where the reservoirs are constructed, is about 1200 feet above sea-level. It is dotted with an immense number of lakes, the total number having been estimated as high as a thousand. Some of the larger of these lakes afford exceptionally favorable opportunities for the inexpensive storage of water. The dams required are low structures, but the area over which the water is raised by them is so extensive that the cost per unit of volume stored is probably the smallest ever yet realized.

" These remarkably favorable natural conditions for the storage of water have long attracted public attention and were made the subject of an able official report by Gen. G. K. Warren as early as 1870. Exhaustive surveys followed at a later date, and in 1881 actual construction was begun. Up to the present date there have been constructed five reservoirs, with an aggregate capacity of about 91,000,000,000 cubic feet, at a total cost of $678,300.*

" The average annual storage of these reservoirs is estimated at about 40,000,000,000 cubic feet, equivalent to about 5200 cubic feet per second for a period of ninety days. This supply is estimated to increase the gauge height at low water at St. Paul, 357 miles below, from 1 to 2 feet.

" The original investigations, embracing the States of Minnesota and Wisconsin, indicated a practicable storage in Minnesota of 95,000,000,000 cubic feet, and in Wisconsin of 79,000,000,000 cubic feet, or a total in the two States of 174,000,000,000 cubic feet. There is probably little doubt that the system could be extended so as to secure a storage of 150,000,000,000 cubic feet in the two States, an equivalent of about 20,000 cubic feet per second for ninety days. From the results already obtained, it is probable that this storage would not cost above $2 per acre-foot. The effect upon the navigable stage of the river would, of course, vary with the locality considered, and would diminish rapidly with the distance downstream. But considering that such an improvement is of the most permanent character, depending only upon the maintenance of the dams for its perpetuity, the above cost cannot be considered excessive when compared with the vast outlay for the mere temporary improvement of these rivers by present methods. A permanent increment of from 10,000 to 20,000 cubic feet per second to the low-water stage of even so large a stream as the Mississippi River is not to be passed over as a matter of small importance.

" The Volga and the Mississippi rivers constitute the only two *systems* of artificial reservoirs yet constructed, and the only ones designed to improve the navigable condition of streams in their natural condition.

" The construction of reservoirs to feed artificial waterways has been resorted to

* See also p. 304 for additional information. For a general discussion on Storage Reservoirs, see "Forests, Reservoirs and Stream Flow," Lieut.-Col. H. M. Chittenden, Corps of Engineers, U. S. A., Transactions Am. Soc. C.E., 1908; Engineering News, October, 1908; and various related discussions and papers.

extensively, particularly in France, and to a considerable extent in this country. Inasmuch as the expenditure of water in canals is a matter of very exact determination, the storage required for this purpose can generally be estimated with great definiteness.

"The construction of reservoirs for municipal purposes is too common a matter to require particular mention. It is sufficient to say that nearly every city in the world of above 100,000 population has storage facilities of greater or less extent to help out its water-supply.

"The principal development of storage reservoirs for irrigation purposes has taken place in Spain, in France and Algiers, in India, and in the United States.

"For such industrial purposes as the operation of factories and the like many rservoirs have been constructed both in France and in this country. They are generally of small capacity, and costly per unit of water stored, but profitable on account of the great use made of the water. Some of these reservoirs serve an important purpose in protecting the valleys below from floods.

"**Effects on Floods.**—Every reservoir built along the course of a stream is, to some degree, a protection against floods in the valley below. The extent of this protection depends, of course, almost entirely on the ratio of its capacity to the flood discharge. A reservoir that can store the entire flow of a stream is an absolute protection against floods for a considerable distance below. It is difficult to propose any general rule for the extent of this control, but, assuming a general similarity of watershed, it would seem not unreasonable to say that it ought to be decisive to at least such a distance below as will give an additional watershed to a stream equal to twice that above the reservoir. This is simply saying that, in the general case, the reduction of a flood wave by one-third of its volume will rob it of its destructive character.

"But in a great many cases this control extends very much farther. For example, in the case of a flood caused by the rapid melting of snows in the mountains, reservoirs below which can impound this flood will protect the entire valley so far as its destructive influence would otherwise have reached. When it is remembered that the volume of a destructive flood is only a part—probably always less than half—of the total flow of a year, it will be admitted that a storage capacity equal to one-fourth of the run-off, well distributed throughout a watershed, will practically eliminate the evil effects of floods in its streams, and supply a percentage sufficient for the purposes of irrigation.

"It is not necessary, though important, that a reservoir should be empty when a flood comes. Even if full, it still moderates the flow of the stream below, the effect varying directly with the superficial area of the reservoir when full, and inversely with the capacity of the spillway. In this respect it acts precisely as does a natural lake. For example, if the spillway of a reservoir or the outlet of a natural lake be of such dimensions as to require a considerable increase in the depth of water to give much of an increase of discharge, every increment of this depth of outlet means also an increment of the same depth over the entire reservoir. A flood passing such a reservoir

will be reduced by the storage resulting from this increment, and before it can produce a full discharge it must fill the reservoir to the necessary height above the bottom of the spillway. A large reservoir is, therefore, even when full, always a perfect protection against sudden floods. In the case of long-continued floods it greatly retards the arrival of maximum effect and gives ample notice of its approach.

"In fact, this is a very important feature of reservoir action, even where the capacity of the reservoir is not sufficient entirely to prevent the flood. It does prevent freshets—that is, sudden floods—and in smaller streams it is often the suddenness quite as much as the magnitude of floods that causes damage and loss of life.

"A reservoir ceases to be any protection if a flood continues long enough to fill it to such a height that the discharge at the outlet is equal to the entire inflow. The same is true of the restraining influence of forests. A sudden and heavy precipitation of short duration, which might produce a severe freshet in a deforested region, would probably experience considerable retardation, and even reduction, if it should fall upon a forest-covered region; but if the rains continue long enough to exhaust the retentive capacity of the forest soil, to fill all the springs and replenish the ground storage, then forests cease to be any protection whatever. In fact, the presence or absence of forests in a vast watershed like that of the Mississippi River is without appreciable influence upon the great floods.

"In the case of floods, which are the results of combinations of discharges from the various tributaries, reservoirs may actually operate to increase the combination. Take for example the natural reservoirs at the sources of the Mississippi. While they restrain the flood excess in that stream, they keep up a heavy flow for some time after the flood has passed, If this larger flow happens to come in with a flood crest at the junction of some tributary below, it will actually increase the combination over what would have been the case without the reservoirs. In the French investigations, presently to be described, the dams proposed for restraining floods were to have open sluiceways without means of closing them. In the ordinary flow of the stream all the water could pass through. But they were to be so proportioned that when the flow should pass a certain point the surplus would be retained in the reservoir, the outflow being always limited by the capacity of the open sluices. The arrangement was, therefore, precisely like that of a natural lake without a dam across the outlet. The outflow could never be entirely restrained, and it would increase in proportion to the height of water in the reservoir. Now, in the case of a large stream like the Rhone, where flood *combination* is the really dangerous thing, it was found that these reservoirs, had they actually been constructed, would have increased certain floods. They would have maintained a heavy retarded flow on some tributaries which in their natural condition would have entirely run out before the arrival of floods from other tributaries. As it happened, this retarded flow in the one case would have come upon a flood crest in the other, and would actually have increased the natural combination. This, of course, could not be

true of reservoirs with closed sluices, unless, as above stated, the reservoirs were entirely filled with the flood passing over them.

"It is, therefore, clear that the efficiency of reservoirs in moderating great floods would have to be a matter of judicious management in controlling combinations quite as much as of actual capacity.

"Another matter to be noted in this connection is that flood protection and industrial use are not entirely compatible objects. To serve the former purpose alone the reservoir should be kept empty until the flood arrives, so that its whole storage may be available. But this might leave the reservoir only partly filled when its supply is needed for other purposes. Generally, therefore, the whole capacity of reservoirs built for these joint purposes cannot be counted on for flood protection. It would probably be unsafe to allow a higher efficiency in this respect than 50 per cent.

"For reasons to be fully considered further on, very few, if any, reservoirs have been built for the exclusive purpose of protecting against floods the valleys below them; but there are numerous examples where this has been an important consideration in their construction. Two cases may be cited in France. The celebrated dam at the Gouffre d'Enfer, on the river Furens, near St. Etienne, was built largely to protect St. Etienne from the destructive freshests of the Furens. It was of course expected to make use of the stored water for industrial purposes, which in a thickly populated district could not but be important. As to the results obtained, the expectations in regard to flood protection have been fully realized.

"The Ternay Dam likewise had as an important motive for its construction the protection of the town of Annonay from the floods of the Ternay, although in this case, as in that just cited, industrial uses of the stored water were considerations of great weight. The result of this work, as to flood protection, has been a success.

"There are certain reservoirs in Germany, as that at Dahlhausen, on the Wappen, and another in the valley of the Bever, which serve very much the same purpose as do those at Furens and Ternay in France, and exercise an important influence upon the floods in their respective valleys. Various similar works have been constructed in other parts of Europe, but all have other motives in addition to that of flood protection to justify their construction.

"The systematic creation of a comprehensive system of reservoirs on any river for the sole purpose of mitigating the severity of floods has never been undertaken. The subject has, however, received exhaustive study, and some examples of such studies will therefore be of importance in this connection. By far the most important of these studies, as might have been expected, is to be found in France. It took place during the reign of Emperor Napoleon III., as a result of the floods of 1856. These floods were among the greatest and most destructive that had ever visited France, and aroused a great deal of interest in the question of their future prevention. Among the various proposals which were brought forward at the time was that of constructing reservoirs

at the headwaters or on the tributaries of the various streams, among which particular attention was given to the Rhone, Garonne, and Loire. These investigations were ordered by the Emperor under date of July 19, 1856, and resulted in the most exhaustive analysis of the whole subject and in reports of great scientific value. They embraced the three streams above mentioned, and the result was adverse to the project so far as the Rhone and Garonne were concerned and favorable as to the Loire. A brief résumé of the reports will here be given.

"*Rhone River.*—The damages wrought by the flood of 1856 in the Rhone Valley were extraordinary. Over 540,000 acres of rich valley lands were submerged and the newly started crops were destroyed. The injury to bridges, dikes, revetments, and other river works was very great, as was also the destruction to the towns and cities situated along the stream. The total damages on French soil in the Rhone valley were estimated at not less than $6,000,000.

"So great a disaster in one of the most populous sections of France naturally led to inquiries into the possibility of preventing a recurrence of it. Napoleon III., who had taken a great interest in public works and favored a liberal extension of them, ordered an elaborate investigation of the subject; first, as to the immediate protection of great centers of population, and second, as to the practicability of 'modifying the régime of great watercourses for the protection of the bottom lands by a diminution of floods by means of reservoirs established near the headwaters of the tributary streams.'

"The first part of the program, viz., the protection of the river towns by works intended to confine the floods to proper limits, was reported practicable at a total cost of about $4,000,000. The second part of the program, viz., the question of reservoir construction, was considered in great detail and with a thoroughness of study which makes it the best existing example of what may be expected from similar works in other localities.

"The river Rhone has a total length of about 447 miles and a watershed of about 36,670 square miles. Three hundred and thirty-six miles above its mouth is Lake Geneva, an immense natural reservoir, with an area of 223 square miles. Below Lake Geneva, at the distances given, the main stream receives the following important tributaries:

"The Arve, $1\frac{1}{4}$ miles below the outlet of the lake, drainage area 2422 square miles; the Ain, 110 miles, drainage area 1355 square miles; the Saône, 131 miles, drainage area 11,019 square miles, the Isère, 179 miles, drainage area 4360 square miles; the Ardèche, 225 miles, drainage area 938 square miles; the Durance, 272 miles, drainage area 5716 square miles. The drainage area of all the other tributaries is about 7200 square miles. The drainage area tributary to Lake Geneva is 2663 square miles, of which 2078 square miles pertains to the Rhone above the lake.

"The flood of 1856 in the valley of the Rhone was practically a simultaneous affair

in all parts of the valley. Only in the upper portions was there any apparent progression. The maximum occurred at the mouth of the Arve thirty-six hours before it reached the mouth of the Ain, 108 miles below; but for the entire remainder of the river the maximum occurred on the same day, with a variation of only a few hours. The causes that led to the flood were therefore operating throughout the entire valley, swelling all the tributaries at once, and in consequence causing a simultaneous elevation of all portions of the main stream.

"The following table shows some of the characteristics of this flood, and gives an admirable illustration of the effect of natural reservoirs in moderating the flow of a stream. It will be observed that the flow of the Rhone just above the Arve indicates a run-off of only 4.3 cubic feet per second per square mile. As a matter of fact, the upper course of the Rhone was discharging into the lake 42,360 cubic feet per second, or 21 cubic feet per second per square mile, which would indicate for the entire watershed above the Arve, including that of Lake Geneva itself, 56,480 cubic feet per second. The storage of Lake Geneva accounts for the difference, and actually reduces the flow of the upper Rhone by about 45,000 (56,480–11,472) cubic feet per second.

| Name of Stream. | Drainage Area. Square Miles. | Discharge. Second-feet. | Rate of Run-off Cubic Feet, per Square Mile per Second. |
|---|---|---|---|
| Rhone above the Arve | 2,663 | 11,472 | 4.3 |
| The Arve | 751 | 24,710 | 31.0 |
| The Ain and smaller tributaries below Arve | 3,777 | 161,674 | 43.0 |
| Saône and smaller tributaries below Ain | 11,264 | 49,420 | 4.0 |
| Isère and smaller streams below Saône | 6,079 | 92,662 | 15.0 |
| Ardèche and smaller streams below Isère | 2,916 | 80,307 | 27.0 |
| Durance and smaller streams below Ardèche | 7,232 | 70,600 | 10.0 |
| Durance to the sea | 1,569 | | |
| Entire river | 36,352 | 490,670 | 14.0 |

"Again, it will be seen that the discharge of the great tributary, the Saône, is at a rate of only 4 second-feet per square mile. Although there is no lake forming a reservoir in this valley, as in that just described, the slope of the lower portion of the valley for 100 miles above Lyons is so slight that floods do not pass off rapidly, but fill up the bottoms over 166 square miles to a depth of 10 feet or more, giving a storage of upward of 50,000,000,000 cubic feet. If the flow of this stream had been as great per square mile of watershed as that of the Rhone above Lyons, without the moderating effect of Lake Geneva, it would have been about 363,000 cubic fect per second instead of its actual flow of about 50,000 cubic feet. Without the storage effects of Lake Geneva and of the Saône valley, the discharge of the Rhone at Lyons would have been about 600,000 cubic feet instead of its actual discharge of less than 250,000 cubic feet. The great influence of these two natural reservoirs in moderating the flood discharge of the Rhone at Lyons is thus clearly apparent, and it is evident that without them the range between high and low water, or the ratio of minimum to maximum discharge, would be

much greater than it actually is. It would not, however, be correct to infer from this that the destructive power of the great floods of the Rhone would, under the above supposition, increase in the same proportion as the discharge itself. Nature adapts the channels of streams to the work required of them, and if the flood flow of this river were greatly increased undoubtedly it would carve out a deeper and wider bed, and would carry away, within the limits of safety, a much larger volume of water than it does at present. Thus, while the absence of these natural reservoirs would, probably, to some extent increase the destructive power of the floods of the Rhone, it would not do so in anything like the same proportion in which it would augment the flood discharge at Lyons.

"When, in the course of their investigations, the French engineers undertook to supplement the effect of these natural reservoirs by artificial ones, they were confronted with practically insuperable obstacles. Nature had not provided suitable localities, and an exhaustive study of the basin gave only the following meager results:

"Lake Geneva could be so dammed at the outlet as entirely to cut off its discharge at the time of flood.

"The Arve and its tributaries, being mostly torrential streams, afford very few good reservoir sites. In fact only one was deemed worthy of consideration, and its capacity was only 706,000,000 cubic feet. This would be of no use to the upper Rhone, which flowed between high banks not subject to overflow, and by the time it reached Lyons its effect would be wholly inappreciable. The reservoir would cost $400,000, besides the destruction of valuable bottom lands. This project was therefore not considered practicable.

"The next site in passing downstream is what is known as the Lac du Bourget, situated to the east of the river and forming a kind of natural reservoir in times of flood. It was proposed to carry this natural action still farther by damming the Rhone. Its natural storage capacity is 3,350,000,000 cubic feet, and this could be increased to 5,824,000,000 cubic feet. It was calculated that this storage would diminish the flow of the Rhone at the moment of flood by 35,000 cubic feet per second, and would diminish the height of the flood at Lyons by 2.3 feet. The cost of this work would be about $4,000,000.

"No further reservoir sites of importance were found above the junction of the Ain. In this valley there are several feasible sites, whose aggregate capacity would be nearly 2,000,000,000 cubic feet. The cost would be about $1,400,000. The estimated effect at Lyons on a flood like that of 1856 would be to reduce the height of the flood by about 1 foot.

"No reservoirs were recommended for the Saône, because none that could be found would have any appreciable effect as compared with the vast natural reservoir formed by the lower part of the valley already alluded to, and would have almost no influence on the discharge of the main stream at Lyons.

"Below Lyons the immediate valley of the main stream offers no opportunities for large reservoirs.

"The first large tributary on this section of the river, the Isère, was carefully studied, but no situations were found which were considered favorable. The alluvial and unsatisfactory nature of the foundation for dams, the necessity of condemning valuable bottom lands, the small aggregate result possible of attainment under the most favorable circumstances, rendered the project wholly unadvisable.

"The valley of the Ardèche likewise contains no feasible reservoir sites.

"On none of the other tributaries were suitable sites found until the Durance was reached. The valley of this stream, which is one of the largest affluents of the Rhone, offers several good sites, and it was found practicable to store 11,366,600,000 cubic feet of water at a cost of about $6,600,000. The result, however, was altogether insignificant. The Durance enters the Rhone far down the valley of that stream, where its flood discharge is already very great. The effect of the proposed reservoirs on the flood of the Rhone immediately below the junction would be to diminish its height by less than 1.3 feet.

"The following tabular summary shows the magnitude and cost of the foregoing works:

| Reservoir. | Capacity, Cubic Feet. | Cost. |
|---|---|---|
| Lake Geneva | 2,294,500,000 | $1,000,000 |
| Valley of Arve | 706,000,000 | 400,000 |
| Lake du Bourget | 5,824,000,000 | 4,000,000 |
| Valley of Ain | 2,000,000,000 | 1,400,000 |
| Valley of Durance | 11,366,600,000 | 6,600,000 |
| Total | 22,191,100,000 | $13,400,000 |

"The result of these works and of this expenditure may be summarized as follows:

"Over the 24,700 acres of submergible lands the depth of overflow would be reduced from 2.2 to 3.2 feet. But this would not entirely prevent submersion, and the necessity for dikes would exist as before. Through Lyons the flood height would be reduced possibly 3 feet, but would save none of the special works of protection and would but slightly diminish their cost. From Lyons down the diminution of height of flood would be as follows: Below mouth of Saône, 1.3 feet; at Tournon, 0.8 foot; at Valence, 0.6 foot; below Valence, inappreciable. The effect of the proposed reservoirs in the valley of the Durance on the floods of the Rhone below the junction of the two streams would be to diminish the flood height at Beaucaire 1.3 feet; at Arles, about 0.5 foot; below Arles, not at all.

"The effect of these reservoirs, therefore, although considerable in absolute magnitude, would not be sufficient, in comparison with their great cost, to justify adoption and the project was reported upon adversely by the engineers.

" This report does not deal with the low-water flow of the Rhone at all, nor with the effect which this storage would have upon the interests of navigation. Undoubtedly it would be much greater than in the control of floods. For example, the 10,000,000,000 cubic feet of water that could be stored upon the upper Rhone and the Ain would provide a flow of about 4000 second-feet for one month, or 1300 second-feet for three months, and could undoubtedly be so regulated as to be of considerable advantage to navigation. The increase for a period of one month only over the low-water flow at Lyons would be nearly 50 per cent.

" *Garonne River.*—Similar studies to those just described were also made in the case of the Garonne, which had likewise suffered severely from the floods of 1855 and 1856. Without reviewing these studies in detail, the following conclusions may be stated in the language of the report:

" ' Reservoirs, when their capacity is great enough, have a very powerful effect in diminishing the flood discharge of the streams on which they are built, but their influence diminishes enormously with distance; and inasmuch as suitable sites can be found only in the mountainous regions, far removed from the bottom lands to be protected, it may readily be seen how slight must be their influence on the flood heights in the valleys far below. . . . To reduce such a flood as that of 1855 to the height required in order to contain it within the proposed system of dikes would require a storage capacity exceeding 33,000,000,000 cubic feet, and would cost $24,000,000. . . .

" ' Other objections of a fundamental character as to all reservoirs have already been stated.

" ' The conclusion arrived at, therefore, is that the idea of reducing the floods of the Garonne by means of artificial reservoirs must be abandoned.'

" *Loire River.*—The studies devoted to this question in the case of the river Loire were more favorable to the use of reservoirs. This was owing to the more favorable conditions which prevail on that stream. The main stream is formed by the union of the upper Loire and the Allier near the city of Nevers at the Bec d'Allier. The Loire is subject to the most extreme variations in the matter of flow. At the junction of the two streams, for instance, it varies from about 10,000 cubic feet per second to 350,000 cubic feet. The floods in the lower river are ordinarily rendered harmless by the arrival, at different times, of the floods from the various affluents, but when the conditions cause the simultaneous arrival of flood-crests from several tributaries the results are liable to be of the most serious character.

" The floods of the Loire River have always been a matter of great moment to the interests of the valley, and have led to extensive works for their control. In the studies above referred to the use of reservoirs on certain portions of the streams was recommended, viz., upon the upper Loire and the Allier. These two streams, heading in the south-central part of France, flow north nearly parallel to each other at distances scarcely ever 50 miles apart. Their drainage areas are 7000 square miles and 4500

square miles, respectively. The geographical, geological, and meteorological conditions are essentially the same for the two streams. They rise in high land some 4500 feet above the level of the sea. The mountain slopes are steep and the soil of a very impervious character. The result is that the run-off responds quickly to the rainfall; floods are quick and of short duration, and the curve of the flood-wave at any point is sharp in character, i.e., very high compared with its length. The conditions in the two valleys are so similar that the crests of floods reach the junction very nearly at the same time, being only two or three hours apart in the great flood of 1856. The curves of discharge of the two streams, both accentuated in character, are superimposed upon each other, producing a curve of relatively the same relief, but absolutely nearly twice as pronounced, as in the case of either tributary.

"The union of two such considerable tributaries with floods of the nature above described gives character to the flood-wave of the united stream for a great distance below, or until the accession of tributaries reaches an extent that may exert a marked modifying influence. But it is stated that the sharp form of the wave does not entirely disappear even to the mouth of the river.

"The flood conditions, therefore, prevailing from Nevers for a long distance down are those of extreme height but short duration. Were it possible to cut off the upper part of this curve and retain the water which it represents, thus reducing the flood curve to the normal form of the other principal tributaries, the floods would be brought within limits which would keep them between the dikes proposed to be constructed along the river.

"An examination of the valleys of the upper Loire and the Allier disclosed the following possibilities as to the storage of water:

"In the valley of the upper Loire twenty-two reservoirs would store about 8,250,000,000 cubic feet of water, and would reduce to 111,653 cubic feet per second the flood-flow, which, without, these reservoirs, would be 153,555 cubic feet per second at Bec d'Allier. In the valley of the Allier sixty-three reservoirs, storing about 10,000,000,000 cubic feet, would reduce to 104,664 cubic feet per second the flood-flow which, without the reservoirs, would be 167,675 cubic feet per second. The total reduction would therefore be about 95,000 cubic feet per second from a total flood-flow of 320,000 cubic feet per second, or a reduction of about 30 per cent. This would deprive floods of their destructive character as far down as to the mouth of the Cher, a distance of about 180 miles below the junction of the two streams.

"It is thus seen that the peculiarly favorable conditions existing on the upper Loire make possible an important reduction of flood-height for a certain length of the river below Nevers. In the upper valleys, near the reservoirs, their effect would, of course, be far greater, and would effectually remove the possibility of flood.

"These proposed works, however, were of great magnitude, estimated to cost over $13,000,000, and they have never been carried out.

"A very interesting and exhaustive investigation, similar to those just described, has been conducted by German authorities in the valley of the river Alb. The study goes into too much detail to be given here, but its general conclusions are so in line with those of the French engineers that they cannot fail to be of interest. The report says:

"'It cannot be denied that for the head waters of rivers, and also for the territory of small streams, the question might be solved. The Government of Würtemberg investigated the matter and found that high floods could be prevented by means of reservoirs, but that the benefit would not be commensurate with the cost. . . . This investigation [the prevention of floods on the Alb] has proved that the construction of reservoirs for the purpose of keeping back the high water of the Alb, although possible, and with no doubt of their effectiveness, is still unjustifiable on account of the enormous cost.'

"And again:

"'There seems to be no doubt that the construction of a system of reservoirs on a large scale in the valley of the Alb is inexpedient, on account of the great cost.'

"Particular emphasis is placed upon these studies, because they disclose the true obstacle to the use of reservoirs for the sole purpose of flood prevention. It is the *cost*, not the physical difficulties, which stands in the way. It may be stated that as a general rule a sufficiently amount of storage can be artificially created in the valley of any stream to rob its floods of their destructive character; but it is equally true that the benefits to be gained will not ordinarily justify the cost.

"The reason for this is plain. Floods are only *occasional* calamities at worst. Probably on the majority of streams destructive floods do not occur, on the average, oftener than once in five years. Every reservoir built for the purpose of flood protection alone would mean the dedication of so much land to a condition of permanent overflow in order that three or four times as much might be redeemed from occasional overflow. One acre permanently inundated to rescue three or four acres from inundation of a few weeks once in three or four years, and this at a great cost, could not be considered a wise proceeding, no matter how practicable it might be from engineering considerations alone. The cost, coupled with the loss of so much land to industrial uses, would be far greater than that of levees or other methods of flood protection.

"In fact, the examples of natural reservoirs already cited, while they show conclusively the vast beneficial influence of large reservoirs upon the flow of streams, also disclose the fatal obstacle to their successful imitation by man. In only very few places has nature prepared sites where man can erect works which will create large bodies of water, and even if she had done so the gain from utilizing them would not equal the loss. The reservoir system of the Great Lakes involves the perpetual withdrawal from agriculture and industrial uses of an area nearly twice the size of the State of New York. Were these areas not covered with water, but occupied as the surrounding country now is, yet so fitted by nature that man, at slight expense, could convert them into great

lakes, as at present, the utter impossibility of such a measure is evident at a glance. And so it will be found in general that the surface of the earth, where reservoirs could be built on an extensive scale, is liable to be of more value in its present condition than it ever could be if covered with water.

"The construction of reservoirs for flood protection is not, therefore, to be expected, except where the reservoir is to serve some other purpose as well, and inasmuch as such purposes are not ordinarily extensive enough to develop *systems* of reservoirs, upon which, rather than upon isolated works, the control of great floods depends, this large control is hardly one of the possibilities of the future. The only probable exception is that of a reservoir system on the watershed of the Missouri River, treated of in the next section of this report.

"For flood protection in isolated cases, however, and on a relatively small scale, reservoirs will undoubtedly continue to be built, particularly when they serve other purposes as well. From this point of view they will always be projects of public importance. The idea is well presented by the distinguished French engineer, P. Guillemain, former inspector-general of public works in France, who holds that the creation of reservoirs is of public utility in nearly all cases, either in flood prevention or in re-enforceing low-water flow, and that whenever special interests, such as industrial uses, irrigation, and the like, exist that will justify their construction, they become legitimate subjects for Government adoption.

"**The Floods of the Mississippi and the Missouri.**—A belief that it is within the range of possibility to diminish materially the great floods of these rivers by means of reservoirs upon its tributaries has long been held. In a work well known in its day (The Mississippi and Ohio Rivers, by Charles Ellet, Jr., published in 1853), the author advocates this view with great vigor, and had his data been as correct as his argument he would have made out a good case. The subject was briefly reviewed by Humphreys and Abbot in their report upon the Mississippi River (1861), and the views of foreign engineers upon this method of river regulation were cited at considerable length. Although the authors of this report pronounced the scheme impracticable so far as the Mississippi is concerned, the idea has, nevertheless, continued to have its advocates from that day to this. It has occasionally found expression in public documents or acts of Congress. In the voluminous report of the Senate Committee on Irrigation, which forms Senate Report No. 928, Fifty-first Congress, first session, the committee say:

"'It is confidently believed that, with restraining dams to hold back the water of the numerous lakes found at the headwaters of the various tributaries of these rivers, and reservoirs constructed at other suitable points, together with the aid of the natural flow of the streams, a very large extent of country, now comparatively worthless, could be made exceedingly productive, while the floods in the lower Mississippi would be greatly alleviated.'

"During the past year two investigations have been ordered by Congress, having

as one of their objects an examination of this reservoir question. Among engineers there are not a few of reputable standing in their profession who hold similar views to those expressed in the Senate report quoted above. With the general public the idea is almost an axiom, and it finds constant expression in the press, particularly when a great occasion, like that of the recent Mississippi floods, calls attention to it. It has therefore seemed important to devote some especial care to the subject, and very soon after taking up the study I arranged to have Mr. James A. Seddon, United States assistant engineer, compile existing data on the Mississippi floods in such form as to present the subject in its entire magnitude so that it can be readily understood.

"Few people have any adequate conception of either the origin or the magnitude of great floods like those on the lower Mississippi. It is a common error to think that they come largely from the melting snows in the mountains.. Yet the floods of the Mississippi nearly all come at seasons when the flow from the mountains is very small. In the greatest known flood of the Mississippi at St. Louis, that of 1844, a large part of which came from the Missouri the latter stream was found by pilots to be in low-water stage above Sioux City. On the occasion of the late heavy flood in the Mississippi, when at its maximum stage, the Arkansas carried practically no water across the Kansas-Colorado line, the Platte did not run above 2000 cubic feet per second at North Platte, Neb., and the upper Missouri and Yellowstone were both in low-water stage. The floods of the Mississippi do not come from this direction. They are formed by the heavy rains in the low regions east of the ninety-eighth meridian, and very largely come from east of the Missisippi itself. The great controlling element, in fact, in all the lower river floods is the Ohio River.

"The magnitude of these floods also depends very largely upon fortuitous combinations of the floods in its tributaries. No single flood from any one of these tributaries, except the Ohio, can produce serious consequences in the main river. But if two or more of them discharge excessive floods in the main stream simultaneously, then it is that great disasters follow. Very fortunately, nature has caused these flood-waves to arrive generally at different periods, and the more disastrous combinations are not of frequent occurrence.

"It is apparent, therefore, that a reservoir system which should exercise any appreciable influence on the lower-river floods must embrace the three great upper tributaries, and particularly the Ohio. What the magnitude of the storage required would have to be may be inferred from the fact that the total discharge of the Mississippi at Cairo, above the bankful stage, during the late flood, was 2,368,000,000,000 cubic feet, or 4250 square miles 20 feet deep, the assumed average depth of reservoirs. The largest artificial reservoir ever built—viz., that at Lake Winnibigashish, Minn.—has a capacity of 45,000,000,000 cubic feet. To store all this excess would take fifty-two such reservoirs.

"While it might seem at first thought that this amount of storage could be found,

still it would be very difficult to find it. Particularly on the upper Ohio and its southern tributaries favorable sites are understood to be of rare occurrence. It is probable, however, that in all the watershed of the Mississippi sites could be found that would insure a reduction of a flood discharge at Cairo like that of 1897 by one-fifth of its maximum. The ease with which the writer was able to find storage amounting to 11,000,000,000 cubic feet in the State of Ohio at the very headwaters of streams along the divide between Lake Erie and the Ohio convinced him that the natural facilities for storage are rather greater than is commonly supposed.*

"As already stated, the difficulty is not so much a physical as a financial one. To store, say, 500,000,000,000 cubic feet of water, equivalent to 11,500,000 acre-feet, would cost even at the rate of only $5 per acre-foot, $57,500,000. This one fact condemns the project as a system for the exclusive purpose of flood prevention. But whenever such reservoirs have other and more immediate purposes for their construction the increment which each will form in the grand total necessary to produce some influence in the Mississippi floods is an element in its favor worthy of consideration.

"The only direct and effective reservoir project, if any such be possible, for impounding floods of such vast magnitude as those of the Mississippi is that pointed out by Mr. Seddon in the second part of his memoir. The project for utilizing St. Francis basin for this purpose would only be following out and perfecting the plan upon which nature has operated for an indefinite period. If the overflow into this basin in a great flood like that of 1882 is equivalent to a depth of 6.5 to 7 feet upon its overflowed area of 6706 square miles, it is at least a reasonable question to ask why this flooded area cannot be reduced to one-half or one-third its present size, be given a depth twice or three times as great, and the water be prevented from flowing out until the following low water. The slope of 120 feet in the length of the basin would seem to make possible a division into separate reservoirs by means of moderate embankments such as Mr. Seddon suggests, making five or six basins of an average depth of 10 to 15 feet, with longitudinal levees to restrict the lateral area. The water thus stored (and it could be stored with such an arrangement, whether there were a high flood or only a moderate one) would give to the lower river in low water an increment (based upon the overflow of 1882) of 141,000 cubic feet per second for a period of one hundred days. This would give a low-water flow of at least 300,000 cubic feet per second, and would radically improve the navigation of the Mississippi from Helena to the sea. From Helena up, the slack-water system through the basin itself, with five or six locks, would carry the deep water to Cairo. How far the imperfectly known topography of the St. Francis basin would lend itself to this project, and whether or not the cost would be prohibitory, exhaustive surveys alone can tell.

"On the Missouri River the case with regard to reservoirs is somewhat different.

* See Report on Ohio Canal Surveys in 1895, House Doc. No. 278, Fifty-fourth Congress, first session, p. 56; printed also in Annual Report of the Chief of Engineers for 1896, part 5, p. 2973 and after.

The annual flood of that stream, which is known as the 'June rise,' is essentially a head-water flood. The earlier floods are generally, although not always, from the lower river, and very rarely from the extreme upper sources. The June rise is the mountain flood, bringing down the snow-water, and generally augmented by the spring rains both in the mountains and on the plains below. Not infrequently it meets with heavy contributions all the way down, and is the result of a general high water over all its drainage area. Ordinarily, however, as already stated, it is a head-water flood, and coming as it does while the banks are still soft and yielding from previous high water, it does its full share of the destructive work peculiar to the Missouri River.

"That a complete system of reservoirs in the mountains and plains portion of the watershed of this stream, which should embrace its many tributaries and contain the waters from melting snows and spring rains, would materially reduce the magnitude of the June rise is highly probable. To take off the flood excesses at Sioux City, mentioned by Mr. Seddon in the first section of his memoir, would require a storage of, say, 48,400,000,000 cubic feet, corresponding to a reduction in stage of 2.8 feet. A storage of 100,000,000,000 would probably give the very material reduction of 6 feet. Allowing a reservoir efficiency of only 50 per cent, as elsewhere explained, and assuming that no one of the great floods of the Missouri has its origin in more than one-half of its watershed, it would seem that a reservoir system of 400,000,000,000 cubic feet, distributed over the watershed above Sioux City, would quite effectually control the floods of the river. This amount of storage is about the percentage of total flow required to be stored for irrigation, as hereafter explained, in order that the water of the arid region may be fully utilized. It must be understood that such a result can be predicted only from a *system* of reservoirs. The effect of any single reservoir would certainly be insignificant, but the combined influence of many might be very important.

"Passing now to the question whether the benefits to the lower river from such a system would be of sufficient importance to justify the construction of reservoirs solely for the purpose of securing them, the answer must be distinctly in the negative. It is still true in this case, as in those already considered, that the benefit is not worth the cost. If, however, there are other and primary considerations, which of themselves would justify the construction of reservoirs, then their influence upon the floods of the lower river is a matter worthy of consideration. And when such primary interests are of a magnitude which looks to a comprehensive system throughout the watershed of the stream, subserving interests of a public as well as a private nature, then the argument for Government assistance in such works stands upon a substantial basis. The point to be especially considered, in connection with such a reservoir system, is that river regulation must always be a secondary motive and more immediate and direct uses the primary motive."

**Ancient Reservoirs.**—Probably the oldest known example of a storage reservoir was that of Lake Moeris, a natural depression lying close to the Nile, about 50 miles

southwest of Cairo. The engineers of the Pharaohs of the twelfth dynasty widened and deepened a dry channel leading to it from the river, and then built across this canal gigantic dikes of earth, which were used as regulators. In times of dangerous floods these dikes were cut, letting the water flow into the lake, which had an area of 695 square miles; after the waters had receded, the dikes were built up again. The cost was so great, however, that they were only opened in times of emergency. The chief object in creating this reservoir was to guard against the excessive inundations which the engineers expected would result from their establishing a system of levees on the right bank, similar to the one built some centuries before along the left bank. As years went by the river, now confined on both sides, gradually widened and deepened its channel, and the time came when the safeguard of the reservoir was no longer needed. The canal was allowed to silt up gradually, and after more than 2000 years of existence, Lake Moeris was reclaimed and converted into the present Fayoum province with its 350,000 acres of cultivated land.*

**Reservoirs of the Mississippi River.**† (See also p. 284.)—The reservoir system at the headwaters of the Mississippi was projected at a time when the traffic by water was of much greater extent than at present. Its object, as described in the foregoing pages, was to assist navigation from the headwaters to a distance of about 40 miles below St. Paul, and to a small extent to assist in controlling floods. There were to be forty-one reservoirs in the system, located in the States of Wisconsin and Minnesota. The project was first advocated about 1867, and the first appropriation secured in 1880. Up to 1909 five had been completed and a sixth was under consideration, the cost to June 30, 1905, including establishment, repairs, cost of damages, etc., having been about $1,500,000. (See Pl. 34.) The cost of establishment only is given as $678,300, equivalent to about $7.50 per million cubic feet of impounded water, on the basis of their estimated storage capacity, and the annual cost of operation and maintenance as about $13,000. The dams were constructed partly of earthen embankments and partly of crib-work provided with sluices, among which were some excellent examples of wooden bear-trap gates. Taintor sluice-gates—also known as sector gates—were also used. The crib-work was replaced later by masonry of concrete and concrete-steel. The combined effect of these five reservoirs, from actual gauge readings, is to raise the stage during the low-water season of summer and autumn at St. Paul, about 150 miles below the last reservoir, from 12 to 18 inches (the endeavor being to keep not less than 5 feet in the channel there), increasing the discharge from about 1500 to about 6000 second-feet. The average effect there during the usual ninety-day low-water period makes a rise of 14 inches. The maximum observed has been 40 inches and the minimum 5 inches. The effect disappears at the head of Lake Pepin, 51 miles below St. Paul. The benefits to navigation

* "The Nile in 1904," Sir Wm. Willcocks.

† Data from Annual Reports, Chief of Engineers, U. S. Army. For information as to the effects of these reservoirs, see the Annual Report for 1906, p. 1443.

and to water powers are said to have been considerable and flood levels have also been reduced, at times by several feet.

The construction of additional reservoirs, with the exception of one at Gull Lake (authorized in 1907), is considered doubtful, as better results (as far as concerns navigation) have been obtained by regularization of the river. The dimensions, etc., of the reservoirs are as follows:

| Name. | Completed. | Height of Dam. Feet. | Maximum Capacity. Cubic Feet. | Cost. | Cost per Million Cubic Feet of Water. |
|---|---|---|---|---|---|
| Lake Winnibigoshish | 1884 | 14 | 45,800,000,000 | $473,900.00 | $5.90 |
| Leech Lake | 1884 | 6 | 30,000,000,000 | | |
| Pokegama Falls | 1884 | 9 | 4,700,000,000 | | |
| Pine River | 1886 | 17 | 7,500,000,000 | 90,400.00 | 12.00 |
| Sandy Lake | 1896 | 9.4 | 3,000,000,000 | 114,000.00 | 38.00 |
| Total | .......... | .......... | 91,000,000,000 | $678,300.00 | 7.45 |

**Reservoirs on the Oder.**—Between 1900 and 1912 a system of reservoirs for flood control was completed on the river Oder in Germany and its tributaries, for the benefit of part of the province of Silesia. The largest dam was placed above the village of Maurer, and was built of masonry, with a storage capacity of about 1800 million cubic feet. The maximum head upon it is 153 feet, and the structure contains 327,000 cubic yards of rubble masonry. The remaining dams were built, some of masonry and some of earth, the greatest length among the latter type being 9800 feet and the greatest head about 57 feet. Power development was arranged for at certain of the dams. The total cost of the works was estimated at $9,500,000.

**Reservoirs on the Nile.***—On the Nile two large reservoirs have been constructed, one at Assuan, the other at Assiut, and a further extension of the system is under way. The purpose of the latter is only partially for storage, and mention of it will be found in the chapter on Bridge Dams. The purpose of the former is to store water for use in irrigation during the months of May, June, and July. The first rise at Assuan usually comes from the White Nile at the end of May, followed later by that from the Blue Nile, the maximum being reached about the middle of September. The average discharge at this stage is given as 353,000 cubic feet per second, the maximum observed being 495,000 feet. The average low-water flow is 14,000 feet per second. To avoid trouble from deposits of silt above the dam, the filling of the reservoir is not commenced till about the first of November, when the water has become almost clear. Until then the sluices in the dam, of which there are 180, placed at four different levels, are left open, and their capacity is sufficient to pass the largest flood without hindering the flow. It is estimated that in an average flood the river during November carries in suspension about

* Proceedings, Inst. C. E., vol. clii.

141,000,000 cubic feet of sediment, and that by gradually reducing the flow, only a small portion of this should be deposited.

The dam was built at the first cataract, about 750 miles from the Mediterranean, and was commenced in 1898 and finished in 1902. (See Pl. 35.) Its total length is 6400 feet, with a maximum height of about 130 feet, and the masonry is of granite throughout and founded on granite bed-rock. About 13,000 cubic yards of masonry were laid in a mortar made of burned clay and lime, ground up with about 10 per cent of sand, and manufactured at the site. This gave excellent results and proved more watertight than a mortar of 4 to 1 of Portland cement. However, as it could not be supplied fast enough, the remainder of the work, amounting to 691,000 cubic yards, was laid in a mortar of 2 to 1 of Portland cement for the foundation bed and the upstream face, and 4 to 1 for the remainder of the work. The maximum pressure on the masonry was limited to $4\frac{1}{2}$ tons (of 2240 lbs.) per square foot, the greatest head on the structure being 65.6 feet. Unusual difficulties had to be overcome in closing the cataract channels with cofferdams, which were formed by throwing boulders into the water until their mass was sufficient to stop the current. In the minor channels the dams were formed of sacks filled with sand taken from the banks and river-bed. Certain portions of the bed-rock, moreover, were found to be rotten, and much expenditure of time and money resulted from having to remove them.

The sluice gates all move vertically, 50 of them sliding, and 130 moving on rollers (Stoney gates). Their size varies from $11\frac{1}{2}$ feet high by $6\frac{1}{2}$ feet wide to 23 feet high by $6\frac{1}{2}$ feet wide, and they are operated by steel cables and hand-power crabs. All the sluiceways are lined with granite ashlar, except the lowest 30, which have castiron linings $1\frac{1}{2}$ inches thick. These were used to permit of more rapid construction. As originally constructed, the maximum pressure on any gate was 210 tons, and the maximum head on the bottom of the gate was about 64 feet, giving an average velocity of discharge of nearly 60 feet per second. After five years' use, during which some of the granite-lined sluices had been discharging for several months each year under a head of 46 feet, it was found that there had been practically no erosion of the masonry, although it was considered too soon to say whether or not repairs might be ultimately needed. Extensive aprons of masonry were added below the dam in 1906, as much of the bed-rock was found to be seamy and unreliable, and considerable erosion had taken place in it.

A canal for navigation was provided, 40 feet wide on the bottom, with four locks, each 263 feet long by 31 feet wide, the average lift between sills being about $19\frac{1}{2}$ feet. The lock-gates were made of iron, from $26\frac{1}{4}$ feet to 59 feet high, suspended from and rolling on overhead girders of counterweighted bascule type, which are swung into a vertical position when the gates are in their recesses.

The structure contained a total of 704,000 cubic yards of masonry, and was built under contract in $3\frac{1}{2}$ years, the amount actually paid to the contractors being $12,250,000,

equivalent to $17.40 per cubic yard. The storage capacity was then 37,612 millions of cubic feet, making the cost about $48.50 per million cubic feet of water impounded.

It had been at first intended to complete the dam, so it would hold water to the elevation of 374.0 feet above sea-level; but as this would submerge certain historic temples, the level was built only to El. 348.0, or 26 feet lower. A few years after the completion of the work, however, its economic value had become so great that the masonry was carried up to about the level originally proposed.

**Reservoirs in Other Countries.**—Many examples of reservoirs on rivers are to be found in India, America, and elsewhere, but as their purpose is for irrigation rather than control of floods, they will not be described.

**Silt in Reservoirs.**—The danger of silting up, which is present to a greater or less extent in every reservoir, does not appear to have caused much trouble as yet in the larger works. In a river where sluices at a low level permit a steady current, as in the Nile at Assuan, the deposit would be slow, and it was estimated by the engineers there that even under the most unfavorable conditions it would be very many years before the capacity of the reservoir would be curtailed, even to a slight degree. Such deposit as might occur could, of course, be removed by dredging. Where, however, a dam is built across a sediment-bearing stream in such a way as to produce a permanent body of quiet water, the upper end of the pool will silt up. The flood current, on meeting the pool, is slackened and begins to let fall its heavier particles, and as it proceeds further and becomes still more checked, the lighter particles are dropped, resulting in a gradual filling of the reservoir from the upper end. These effects can often be traced in mill-dams built across creeks; the upper ends of their ponds are shallow and flat, and often overgrown with willows, and the water gradually deepens as the sluice-gates are approached. The reservoir of the Austin dam in Texas, completed across the Colorado River in 1893, had so filled up by 1900 that 48 per cent of the original contents of the lake consisted of mud. The storage capacity, which had been 83½ million cubic yards, was reduced in seven years to 43½ million yards. The depth of deposit varied from 1½ to 29 feet, most of it being an impalpable silt, the upper two miles alone being of sand.* This dam was built as a solid masonry structure without any scouring outlets, and was of very small storage capacity compared with the discharge of the river, which carried about ¼ of 1 per cent of silt. The flow was sufficient to have filled the reservoir about 40 times per annum. The Sweetwater reservoir in California, however, also a solid structure, into which flows the Sweetwater River, carrying about ½ of 1 per cent of sediment, silted up between 1887 and 1900 at a rate which it was estimated would not fill the basin for more than three hundred years, as its relative capacity was large. An authority recommends in view of these facts that a reservoir on a silt-bearing river should have a capacity not less than the normal annual discharge.†

* "Engineering News," February, 1900.
† Ibid., June, 1900, Jas. D. Schuyler.

**Regulation and Reservoirs.**—The principle of augmenting the low-water flow of a river by using reservoirs in connection with regulating works has been recently applied to the Weser and the upper Oder, where the system is expected to serve the needs of traffic while being much less expensive than canalization. On the Oder the estimates of cost per mile amounted to $60,000 as compared with $140,000 for canalization, the minimum depth of channel being assumed as 4½ feet. A brief description of the reservoirs has been given on p. 305. The effect on navigation of the reservoirs of the Volga has been described on p. 288, and of those of the Mississippi on pp. 289 and 304.

## CHAPTER VIII.

### THE IMPROVEMENT OF THE OUTLETS OF RIVERS.

**Classes of Outlets.**—The outlets of rivers may be divided into three general types: (1) those which discharge directly or indirectly into seas where the range of tide and the violence of storms is limited, such as the Danube, the Nile, the Mississippi, certain rivers flowing into the Baltic, etc.; (2) those which discharge through estuaries, such as the Thames, the Seine, or the St. Lawrence; (3) those which discharge directly into oceans and are exposed to all the changes produced by sand-drift, tidal effects, etc., such as most of the rivers of the Atlantic and Pacific coasts of the United States. Of these three classes the first is usually the least complex in improvement, since the forces at work are comparatively steady, and the complications due to storms, shifting channels and similar phenomena, are less in evidence than with the other two. The second type appears for the same reason to be in some ways more simple than the third, since the shores of the estuary afford some protection from the violence of storms, and usually permit the maintenance of a more stable channel. The length of the latter, however, is frequently very great and its improvement correspondingly expensive.

**Formation of Bars.**—Various causes contribute to the formation of the bars which are found at the mouths of almost all rivers, and which are usually an impediment to navigation. With sediment-bearing rivers, like the Mississippi, Nile, Amazon, etc., and other rivers entering lakes or seas directly, the current on meeting the still water is checked in its flow and the sediment held in suspension cannot be carried further, and the result is the formation of a bar. The bars at the outlets of lagoons or bays which empty into tidal seas and which receive the flow of a river, as Galveston and Mobile bays, are created chiefly by the action of the winds and waves in driving material into and across the mouth. If it were not for the tidal currents the mouths would in consequence become closed and the shore line be made continuous by the action of the wind and waves. In such cases very little, if any, silt from the river reaches the bar, and such of it as does, being very light, is carried away and deposited upon the shore or in deep water.

When a river discharges through a tidal estuary, the bar may be due to the conflict of the ebb and flood currents at the outfall, causing eddies and still water; to the difference in their duration and consequent scouring action; or to the action of waves or sand-drift along the shore, resulting from prevailing winds or currents.

**General Principles.**—The operation of the laws governing the formation and the improvement of the outlets of rivers and tidal harbors is usually complex and difficult of any close analysis. The forces at work are generally many and varied, and while the effect of a single one upon a plan for improvement might be foretold, their action in combination can only be approximated. There are, for example, as just mentioned, the transportation and deposit of sediment, present in most rivers; the effects of floods and tides; the presence or absence of currents along the coast; and the gradual effects of storms and the drift of shore material, which with small rivers may change the outlet entirely, as with the river Yare on the east coast of England, where the outlet was driven south 4 miles in the course of years, and at Aransas Pass in America, which has moved to the southwest about a mile in the past fifty years. In some cases such causes produce daily changes of channel, as with the Hoogly, where ships can navigate only in daytime and by constantly taking soundings; in others the movement is more gradual, as with the estuaries of the Thames and the Seine; in others again the movement may occupy many years. Occasionally other forces are met with whose causes and effects are very obscure. Thus at the harbor of Madras, during the construction of the jetties, certain portions of them, in a depth of 18 to 20 feet of water, were frequently covered 3 feet deep with sand during seasons of calm weather and with no perceptible littoral current.*

It will often be found, however, from a close study of the charts of various periods and other data pertaining to the locality in question, that Nature appears to have indicated certain persistent tendencies which the river or estuary is following in making its way towards and across its bar, and which may afford valuable assistance in studies for the improvement. In difficult cases an investigation by a model after the method indicated by Mr. Vernon-Harcourt, described further on in this chapter, may prove of value in removing some of the uncertainty from the more obscure features.

Mr. Vernon-Harcourt has laid down the following principles for improving tidal rivers, which, it should be noted, are not general, but apply more particularly to streams entering tidal estuaries.

"(1) The tidal flow should be admitted as far up a river as possible, and all barriers to its progress removed, so that the period of slack water may be reduced to a minimum. By this means also the area of inevitable deposits is enlarged, and thus the deposit does not unduly shoal the channel when the fresh-water discharge is small, and the volume of tidal water flowing through the outlet is thereby increased.

"(2) The fresh-water discharge should not be abstracted, if possible, for supplying canals or for other purposes, but should be directed into the upper end of the main tidal channel, so that it may have the fullest possible effect in reinforcing the ebb throughout the whole of the tidal course of the river, as the power of the outflowing current to maintain the channel depends upon the additional force thus furnished to the ebb.

* Proceedings, Inst. C. E., vol. C.

"(3) The form of the estuary should be regulated so as to enlarge gradually as it approaches the sea, and thus promote regularity of flow without unduly restricting the tidal capacity above the outlet. This may be sometimes accomplished by low training banks which, whilst directing and concentrating the latter half of the ebb, do not materially impede the admission of the flood-tide up the estuary. Where the estuary is very wide and irregular, and the main river channel through it is very tortuous and shifting, high embankments may be formed, on each side, widening out toward the sea, and the land behind them reclaimed."

The principles governing non-tidal rivers and those where the range of tide is very small, differ to some extent from those given above because of the lack of tidal influences capable of affecting the maintenance of their outlets, and because of the difference in form of the mouths themselves.

In this class of stream the current always flows in the same direction—toward the sea—upon reaching which it is suddenly checked, and, as before stated, deposits its load of sediment. This gradually builds up a bar which forces the water in various directions through separate outlets across the foreshore, and forms what is known as a delta. This division into various arms or outlets tends to reduce the scouring effect of the current, and the channels become too shallow for navigation by reason of the deposit of the matter brought down by the river. These deltas gradually extend into the sea as this material is progressively deposited at the mouths of the outlets.

The following principles are laid down by the authority just quoted for improving non-tidal outlets, without littoral currents, or where these currents are very slight.*

"(1) The only method of deepening the outlet of sediment-bearing rivers flowing into tideless seas is to prolong one of their delta channels by parallel jetties out to the bar, so that the prolonged current, being concentrated across the bar, may scour a deeper channel, and carry its burden of sediment into deep water further out.

"(2) One of the minor outlets should be selected for improvement, if its delta channel is adequate, or can easily be made adequate for the requirements of navigation; and the discharge of the other outlets should not be interfered with. The advance of the delta at one of the minor outlets is slower, and the distance out to the bar is less, and consequently the jetty works are less costly; whilst an increased discharge, produced by impeding the flow through the other outlets, would also increase the volume of sediment, and therefore quicken the rate of advance of the delta, and hasten the necessity of prolonging the jetties.

"(3) The success of the jetty system depends on a rapid deepening of the sea in front; on the fineness and lightness of the sediment brought down; and on the existence of a littoral current, its velocity, and the depth to which it extends. Any erosive action of winds and waves along the shores of the delta is favorable to the system, and also any reduction in density of the sea-water, such as may be found in an inland sea.

* "Improvement of the Maritime Portion of Rivers."

"(4) If the sea-bottom is flat; if a large proportion of the sediment is dense, so that it is carried along the bed of the river or close to it; if the outlet faces the prevalent winds; and if no littoral current exists, it is possible that an improvement of the outlet may not be practicable; and then recourse must be had to a side canal, starting off from the river some distance up, and entering the sea beyond the influence of the alluvium of the river.

"(5) The bars in front of the outlets of tideless rivers being formed by the deposit from the river, vary in form according to the nature of the sediment brought down. When the material is composed of particles of very variable density, it is gradually sifted as the velocity of the current decreases, and gives a flat sea-slope to the bar. When, on the contrary, most of the material is heavy, the bar has a flat river-slope, as in the first case, formed by the gradual arrest of the sediment rolled along the bottom; but as little of the material is carried beyond the crest of the bar the sea-slope is steep.

"(6) The jetty system does not constitute a permanent improvement, for, sooner or later, in proportion as the physical conditions are unfavorable or the reverse, a bar is formed further out, and a prolongation of the jetties becomes necessary."

The last rule would not apply if there were a prevailing wind which caused a littoral current sufficient to carry away the silt as fast as it was brought out by the river.

A decision as to the most advantageous plan to follow must of course be based on a careful study of the tendencies and effects of the natural forces at the particular locality as well as of the results of corresponding improvements elsewhere, since, as with the regulation of a river, fixed rules are not applicable to all cases, and judgment must rather be based on the development of general principles.* The conditions at an outlet, moreover, are often complicated by the shifting of the channel, due to the drift of sand along the coast or to disturbances produced by storms which, in an exposed outlet, may block up a channel and cause a new one to open in a very short time. Where the effect of the littoral drift is small, the general tendency of the flow from the outlet appears to be along the shortest path to deep water, this being under ordinary conditions the line of least resistance, and frequently nearly at right angles to the adjoining coast. In many cases there exist two main channels to the bar (in addition to the small side or "swash" channels) which deepen and shoal alternately during a cycle of change, and shift their location within a sector covering an angle usually from 45 to 90 degrees. This cycle of change may be illustrated by taking as an example an outlet whose limits of change lie between southeast and northeast, and whose main channel for the time being is the southerly one. After this channel has continued in existence for perhaps some years, it will begin to shoal,

* For a description of the principles and methods evolved from experience in America, see "Harbor Improvement on the Pacific Coast of the United States," by Major Wm. W. Harts. Corps of Engineers, U. S. A., published in the "Professional Memoirs" of the Corps, October–December, 1911, and also in the Proceedings, Inst. C. E., and for a general discussion on outlet and harbor improvements, see Transactions Am. Soc. C. E., vol. liv., Part A, 1904 (International Engineering Congress of St. Louis). Additional information will be found in various papers of the International Navigation Congresses.

possibly from a single gale or from a period of gales, possibly from more obscure causes. At the same time the northerly channel will begin to open, and the closing of the southerly one will continue until it has become valueless for shipping. Usually more or less change of intermediate location occurs during this period, the channels sometimes wandering over a considerable portion of their field before the final shoaling or opening occurs. The northerly channel will then pass through a period as did the other, and may shift further north, gradually deteriorating until the natural forces close it, and the water breaks open again along its first direction towards the south. Observations are few regarding the regularity or duration of these cycles; at some outlets a change as just described has taken place in a few years' time, at others the channels have remained almost stationary for a very long period.

The resultant or magnetic direction of a channel across its bar, however, often appears to show a more or less constant tendency in spite of its general changes of location, and where this condition is found it appears to indicate a natural balance of local forces of which account should be taken in projecting an improvement. Thus the bar crossing of the Columbia River in Oregon is stated to have always shown a marked tendency towards a southwest course, and that of the South Pass of the Mississippi, towards a southeast course.

**Advance of Bar.**—Experience seems to show that the advance of the bar after improvement is rarely a serious factor where the works have been well designed and rapidly carried out. The pockets formed by the jetties on the outside offer receptacles in which the sediment or sand-drift gathers, sometimes behind one jetty only, the other one showing erosion, sometimes behind both. When the depth of water is great, the sediment has a wide field for deposit, and the bottom builds up slowly in consequence. This question, prior to the improvement, caused an apprehension of serious trouble at the mouth of the South Pass of the Mississippi, where it was claimed that the jetties would have to be advanced into the Gulf every few years. In point of fact, however, no extension has been made, nor found desirable. Most of the vast amount of sediment brought down has been deposited to the east and west of the jetties, and the river has never failed to maintain a channel to the open sea. (See Pls. 36, 37 and 38.) Mr. Vernon-Harcourt states that at the harbors of Sunderland, Ymuiden, Dublin and Port Said, the advance of the foreshore proved to be much less than was anticipated, and at the Sulina mouth of the Danube, which discharges a great amount of sediment, Sir Charles Hartley stated that the rate of advance of the bar, according to observations taken for ten years after the completion of the works, had been reduced by the improvement more than one-half. (Pl. 39.) With the St. John's River, and at other localities where the improvement was retarded by lack of funds, the eroded material and the sand-drift appear to have followed closely the gradual extension of the jetties. (Pl. 40.)

**Littoral Drift.**—Where waves break in a considerable depth of water, or where the outside currents flow in a similar depth, the bottom appears to be slightly, if at all, dis-

turbed; but where the depths are shallow, the waves and currents will stir up and transport the material. Tests made in 1902 at Cumberland Sound showed that coarse sand and shell, when stirred up by breakers, were carried to a considerable distance even by light currents, and were not deposited till smooth water was reached. The same materials in quiet water lay undisturbed by currents flowing as swiftly as 4 feet per second, although fine sand was found to be moved by comparatively light currents. This action, on exposed coasts, leads to a constant movement of the sand or shingle, and if the storms prevail in one direction, there will be a corresponding littoral drift. Where jetties or breakwaters are built under such circumstances, there will result an erosion on the lee side and a filling on the windward side, and this will continue until the latter is rounded out and the sand can travel past the ends of the jetties and continue its movement along the coast. The construction of breakwaters for the harbor of Madras led to an erosion of the neighboring coast for a distance of several miles to the north, in which whole villages were destroyed, and at the harbor of Ceará in Brazil, a similar erosion took place and continued for about three years, until the littoral drift had silted up the windward side and the entrance, and could pass along as before.* At the mouth of the St. John's River in Florida, the beach to the south was similarly eroded. (See Pl. 40.)

**Methods of Improvement.**—There are four general methods in use for improving navigable conditions at the mouths of rivers: (1) by lateral canals; (2) by dredging; (3) by jetties and dredging combined; (4) by jetties only.

*Lateral Canals.*—Under favorable conditions lateral canals afford a solution of the problem so far as concerns vessels of small draft; but their locks cause some delay to navigation, and if the canals are intended for deep-draft vessels, the problem of maintaining the outlet to deep water is again met with. Those cut at the outlets of the Rhone, the Nile, and the Tiber, by the ancients, were mere derivations without gates, and the sediment of the rivers entered at flood time and bars were formed at the sea extremities. Those of the present day usually have locks connecting them with the river, so as to control the variation of tide. The estuary of the Seine offers an example of an outlet where a trained channel and a lateral canal are both employed. Seaworthy craft in proceeding to or from Havre use the freer but more exposed channel of the open water, while the river boats use the lateral canal, which leads from Havre to the upper end of the outlet at Tancarville.

*Dredging.*—Where the improvement is to be carried out by dredging, the line of excavation should be located to conform as far as possible to the natural trend of the current, which will then assist in opening and keeping clear the new channel. If the location is made oblique or across the current the excavation will be constantly filling again, and will entail much work to keep it open. This method has been found an economical means of improvement where the cost of jetties would be too great. Thus the bar of the Mersey, a third of a mile wide but with deep water on each side, and which had only 11

* Proceedings, Inst. C. E., vol. clvi.

feet of water in the sailing channel at low spring tides, with a tide range of 30 feet, was dredged until there was from 24 to 28 feet at low water. This improvement has been secured and maintained by suction hopper-dredges, which commenced working in 1890. In 1910 the dredging was increased with the object of securing a depth of 35 to 40 feet at low water.

The entrance to the harbor of New York, the channel across the bar of the Loire, the entrance to the harbor of Port Natal, and many other localities are maintained by this means. At the last-named place the channel when cut through is reported to have maintained itself for some years, and even after heavy gales very little shoaling was found to have taken place.

This method of improvement has been very largely applied to river and harbor outlets in recent years, as results are often obtained in much less time than would be the case with jetties, and the annual cost of the dredging is in many instances considerably less than would have been the annual interest on the cost of construction if jetties had been employed. Moreover, the demands of commerce for increased depths of water frequently render too expensive any other method than dredging, or dredging combined with jetties.

*Dredging Combined with Jetties.*—Dredging combined with jetties or training walls is also met with in many instances, as in the estuaries of the Seine and the Clyde; the mouth of the St. John's River, Florida; the Mississippi; the Columbia River, Oregon; the Sulina mouth of the Danube, etc. It has proved to be a very efficacious means of hastening the formation of the new channel, and of maintaining or increasing the depth after the scouring force of the water has reached its limit. Additional information in regard to this class of dredging has been given in Chapter III.

*Jetties.*—The object of jetties is to confine and thereby concentrate the flow across the bar and produce scour there, and, with shifting outlets, to protect and make stationary the location of the channel. The great depths required by modern commerce, however, are rarely obtainable solely from the scouring action resulting from such concentration, or if so obtainable are usually too long in developing. Hence in the important locations where jetties are employed dredging is now almost invariably used as an adjunct in producing and maintaining the required depth.

In some cases, as with the South Pass of the Mississippi (Pls. 37 and 38), or at Tampico (Pl. 44), the natural conditions permit the jetties to be placed parallel and to form the approximate limits of the new channel; in others, as with the St. John's River, Florida, (Pl. 40), or the projected system of the Columbia (Pl. 43), they commence at the nearest shore line and converge at the bar. In the latter case the river usually shows a tendency to fill up more or less of the shallow places inside the jetties, but outside of its general current, in the same way as a delta-forming river gradually builds up and extends its banks into the sea. The width between the outer ends of the jetties is determined as nearly as may be from a study of the discharge and the velocity of the current, and

from an examination of the natural conditions and tendencies of the outlet. It is important to carry the ends of the jetties far enough from the shore to prevent the encumbering of the outlet by the sand stirred up by storms and drifting along the coast, as if this precaution is neglected the current may be unable to keep the bar channel open as desired. According to observations by English engineers waves will not stir up sand to any noticeable extent where the water is 24 feet or more in depth, unless some special cause creates a strong reaction,* and experience in America seems to indicate that in order to avoid trouble from this cause the jetties should be carried out to a depth of at least 20 feet of water, and to considerably more if on an exposed coast, in order to prevent the movement of sand along the bottom from encumbering the mouth.

With the Galveston jetties it was found the bar was pushed seaward with the advance of the works, keeping pace with them as long as the depth of water remained shallow, the material eroded by the current being replaced by the local forces of the waves, etc. It was only when the jetties reached deep water that the scour began to show effective results, and then its work was rapid and has remained permanent and self-sustaining.

When double jetties are to be used and are not built simultaneously it is considered preferable to build first the one on the side from which the littoral drift comes, as this will tend at once to protect the new channel from the deposits and enable the scour to begin effectively.

In wide, sandy estuaries, where the channel is often crooked, the currents are usually trained within fixed limits, gradually enlarging into deep water. This facilitates the tidal flow and also the discharge of the river in such a manner as to render maintenance less difficult. The distance apart at which these training-banks should be placed, as well as the height to which they should be built, has ever been a fruitful source of discussion. Vernon-Harcourt states that the training of a river by longitudinal banks always conduces to the improvement of an irregular and shifting channel, and gives as instances of successful works of this kind the Seine, Maas, Clyde, Tyne, Tees, and the Fen rivers. Their location necessarily depends largely upon the natural tendency of the river and the situation of the towns along its banks. The increase in the width between the training walls of the Seine estuary averages 1 foot in 200 feet; of the Weser, 1 in 71; in the Clyde and Tyne, 1 in 100; and in the Scheldt, 1 foot in 50 feet. Such walls usually cause deposit behind them, and in many cases the land thus reclaimed becomes valuable for agriculture, wharves, etc. The natural dimensions of the river often give indications as to the varying distances to be used between the walls. In the improvement of the mouth of the Weser these dimensions were made the basis by which the channel was defined, and the width between the walls was increased gradually in close proportion to the natural effective areas.

In a few cases an attempt has been made, as with the Columbia River, the first improvement at Galveston Harbor, and elsewhere, to form a channel by using a single jetty;

* Proceedings, Inst. C. E., vol. clvi.

but experience with this method, as well as results obtained where one of two projected jetties has been built first, does not appear to have been satisfactory. The single jetty appears to leave too wide a space over which the current can escape, so that the velocity and effect of the latter on the bar remain almost unchanged. Moreover, where the main current runs close to such a jetty, the width of the resulting channel, if the latter becomes deep, is liable to be too small. One example of a temporarily successful single jetty was that of the Lido entrance to the harbor of Venice. The channel depth on the bar was between 8 and 9 feet, and two curved converging jetties were projected so as to decrease slowly and regularly the width of channel till the bar was reached. Owing to the fears of the authorities lest the works should prove a failure, the north jetty was built first and carried out to 28 feet of water. The channel, though not restricted as to width, commenced to scour, and reached a depth on the bar of 20 feet, at which time the construction of the southern jetty was commenced, and by 1905 a least depth of 24 feet had been obtained.*

It is important in improvements by means of jetties that no undue delay be allowed in their construction, as with shifting channels the tendency to movement will continue until stopped by the works, and if these cannot be advanced as rapidly as is desirable, the flow may get beyond control and leave the jetties to one side. An instance of such a risk occurred during the improvement of Cumberland Sound, on the southern coast of Georgia. This work was commenced in 1880 and, owing to the failure of Congress to provide sufficient funds, the construction had to be prolonged over a period of twenty-two years, but was finally carried to a successful completion in the face of unusual obstacles. In 1880 the main or northerly channel ran to the bar about east-northeast, between the projected location of the two jetties. The bar had from 11 to 13 feet on it, with a mean tide range of about 6 feet, and the improved depth was to be 19 feet. The channel, after work was begun, commenced a period of change and began to move southward, so that in 1886 there were two channels, one running about east and the other about southeast, and the engineers drew urgent attention to the need of prompt extensions of the jetties in order to stop the shifting of the flow. The appropriations, however, were still unavoidably small, and by 1892 the sailing channel had swung to the south, crossing the end of the unfinished south jetty about 7000 feet from its shore end (work there having to be suspended in consequence), and thence running on the outside of it almost due east to the bar. This condition of affairs continued for some years, the space between the seaward portions of the jetties, which were then considerably below low tide, gradually shoaling up, and the outlet of the sailing channel swinging further to the south, till about 1896 it reached its limit and began to shoal, a northerly channel having shown signs of opening along the north jetty during the previous year. Apparently, however, this was inadequate for ships, which had to continue to use the southerly channel. By 1898 the latter had returned to an easterly course, almost parallel to, but still outside of the jetties,

* Minutes of Proceedings, Inst. C. E., vol. C., p. 78.

while the channel inside the jetties was a shallow outlet passing to the north of a sandbank which stretched across two-thirds of the enclosed space. So serious was the condition of affairs that the port was almost blockaded, and a portion of the south jetty at the channel crossing had to be removed and dredging resorted to in order to relieve the urgency of the case. In 1900, however, a new channel between the jetties began to open, the ship channel still crossing the south jetty, but slowly shoaling. Dredging was commenced in the new channel, and proved of material benefit, and by 1902 there was depth sufficient to permit the south channel to be abandoned, and the gaps in the jetty to be closed. This final channel was reported to be satisfactory and of good depth and width, although, owing to the comparatively shallow depths between the ends of the jetties (19 or 20 feet), the bar crossing was shifting. By 1910 there had been secured a channel depth of $24\frac{1}{2}$ feet at low water, extending over a minimum width of 400 feet.*

The height above water adopted for jetties is variable, some engineers placing the tops at low tide level, others at mean tide, and still others at high tide. In the United States experience seems to sanction a mean-tide height for the main portion of the jetties, the shore ends often being built to high tide so as to assist in holding the littoral drift. At exposed localities the outer ends almost invariably become battered down below low tide, and cannot be held up without much expense. (See also p. 319.) Training walls in estuaries are usually built to several feet above mean high tide, so as to accelerate deposits behind them.

**Construction of Jetties.**—Jetties of rubble stone, or of concrete blocks, such as are used for all permanent work exposed to wave action, are usually founded on mattresses or fascines, or on a thick bed of spalls or rip-rap, if built on silt or sand, in order to prevent scour and undue settlement. Sometimes a cribwork of logs or plank is used where brush is too costly. Where exposed to sea-water these foundations must settle into the bottom or become covered with deposit, or they will be destroyed by the teredo, and if they are to remain uncovered they should, if practicable, be formed of creosoted materials. The latter method, however, is a temporary protection, as the creosoting has been found to leach out and gradually disappear. These foundations should be kept 100 feet or more in advance of the main body of the jetties, in order to prevent too much erosion by the currents around the ends.

Recent experience appears to show that better results are obtained, where the foundation is of sand, by omitting mattresses, and employing an apron composed of rock ranging from spalls to pieces of the size of a man's head or a little larger. This apron is generally constructed for a considerable length in advance of the superstructure, and is usually made of a sufficient size to support the full width of the latter. It has been observed that the small rock tends to sink into the sand about 2 or 3 feet; but if the apron is made 3 or 4 feet in thickness no further subsidence is usually caused when the heavy rock of the jetty is placed thereon. Experience has shown also that if the currents are

* Annual Report, Chief of Engineers, U. S. Army, 1897, p. 1534, and later.

strong the riprap bed should extend at least 10 feet on both sides of the main jetty section in order to keep the scour from undermining it.

In the Gulf of Mexico it has been found desirable, because of the teredo, to avoid the use of all brush or timber under jetties except where the water was comparatively fresh, as at the outlets of the Mississippi. In some instances there was also great scour under the mattresses before they could be covered with rock.

If the mattress is of small brush, the teredo does not appear to do much damage, except at the thick ends of the pieces. The twigs, however, usually break off in a strong current. While a settlement often occurs during construction due to the scour of the water passing through the jetty under head from the contraction, further settlement may also take place after completion. The view has been advanced that with a high jetty this may be due in part to the hammerlike action from the sudden change of specific gravities as the waves rise and fall. Thus the Kurrachee break-water subsided two feet from the effect of storms almost uniformly and without displacement of the blocks of the superstructure, and the settlement was supposed to be largely due to such a cause.*

Jetties exposed to waves are usually made entirely of loose rubble, with flat surface slopes, or of a rubble hearting with the exposed surfaces protected by large loose blocks of rock or of concrete. In some cases the crown is surmounted by a solid wall of masonry. Where the normal wave-action is not violent, as in the northern portion of the Gulf of Mexico, a weight of 2 to 3 tons each has been found sufficient for such blocks near the shore, while 4 to 5 miles out, and in 30 feet of water, a weight of not less than 10 tons has been found necessary. An example of recent construction is shown by Fig. 128*a*. At the outer ends of the jetties at the Sulina mouth of the Danube, which were subjected to rough seas, blocks weighing 10 to 20 tons were used. At the outer end of the Galveston jetties the covering blocks weighed from 10 to 15 tons while along a portion of the south jetty a continuous crown of mass concrete was placed instead of cap blocks. This crest resisted the storm of August, 1908, while on other portions of the jetty cap blocks were torn off. On portions of the jetty of the Columbia River (Oregon) the covering blocks, weighing from 6 to 16 tons, have often been displaced by storms. At the harbor of Madras, where the works were exposed to heavy gales, the blocks, which were loose, weighed 27 tons each. In unusually exposed situations the superstructure is often built as a solid wall.

It is stated that the effect of ordinary waves ceases to disturb foundations of riprap at depths from 6 to 16 feet below the water surface, according to the exposure, but severe storms or unusual conditions greatly increase the force. Thus at the Alderney breakwater, in the English Channel, where the wall of masonry surmounting the rubble base caused a violent reaction, the foundation was displaced at

* Minutes of Proceedings Inst. C. E., vol. xxxvi.

a depth of 20 feet below low water, while at Peterhead (Scotland) concrete blocks weighing 40 tons have been displaced at depths of 36 feet below low water. At Coos Bay, Oregon, where the jetties are exposed to heavy seas, their outer ends, composed of blocks of sandstone weighing from 2 to 10 tons each, were twice built up to a height of more than 20 feet above low tide, but the crests were eventually beaten down to 10 or 12 feet below the low tide level. At the Sulina mouth of the Danube the waves beat down the outer ends of the first jetties to a depth of 14 feet below water, while nearer the shore the disturbance ceased at 3 to 4 feet below. The stones, however, were small, comprising less than a cubic foot apiece. With jetties on the Gulf of Mexico the ordinary effect of waves on riprap has been found to cease at about 12 feet below low tide.

Where a base of ordinary riprap has been used, it has usually been found safe to construct it with side slopes of 1 to 1 or 1½ to 1 (according to the specific gravity of the pieces) as far up as the expected limit of disturbance. If the riprap

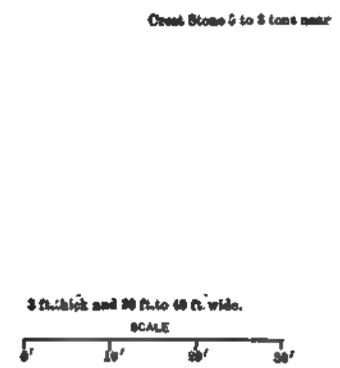



FIG. 128a.—Section of South Jetty, Aransas Pass, Texas (1911).

be continued above this limit without the protection of heavy blocks, the waves will tend to flatten the slopes gradually to 1½ or 2½ horizontal to 1 vertical or less, an extreme having been reached in the case of the Alderney breakwater of 7 to 1.

Where jetties are less exposed to destructive forces, they have been made, according to the requirements, of a close row of round or sheet-piles; of a double row of piles filled between with brush fascines and weighted with stone; of cribs of round or sawed timber filled with riprap; or of a simple dike of riprap.

**Experiments with Models.***—About 1887 a series of experiments was commenced with models by Mr. Vernon-Harcourt in connection with the various plans proposed for the extension of the training works in the estuary of the Seine. For this improvement fourteen different methods had been proposed. A few years previously a model of a part of the estuary of the Mersey had been experimented with by Mr. Osborne Reynolds in connection with works proposed, but never executed, for

* Proceedings, Royal Society.

the outlet of the Manchester Ship Canal, constituting, it is believed, the first investigation of this type.

The model for the Seine was moulded in Portland cement to the scales of 1 to 40,000 horizontal and 1 to 400 vertical, and extended from Martot, the lowest of the dams, to the open sea, its length being about 9 feet. The rise of the spring tides at the entrance, about $23\frac{1}{2}$ feet, was represented on the model by an elevation of only 0.71 inch. Where the natural bottom was rock, the cement was moulded to the exact depths shown on the chart of 1880; in other places it was kept below the greatest depths shown and covered with sand.

To represent the conditions of fresh-water flow, the requisite amount of water was admitted at the upper end of the model through a faucet, and its escape was similarly regulated at the lower end, the water passing out during the higher half of the tide. To represent the tidal flow, which enters by two main channels at slightly different times, a zinc tray was used, hinged by a rubber strip to the model and at the same elevation, and set at the requisite angle to correspond with the inflow of the main body of the tide. The rise and fall of the water was secured by raising or lowering this tray by a screw. When sand had been placed in the model to the average level of the bed, enough water was introduced to represent the surface of the low water of the spring tides, the tray being down. The tray was of such size that when fully raised it poured enough water into the model to represent the high water of the spring tides.

To find the proportionate tidal period, the tidal velocity was first taken as being proportional to the square root of the vertical scale. The latter was 1 to 400, so that the tidal velocity would be 1 to $\sqrt{400}$, or $\frac{1}{20}$. The horizontal scale, 1 to 40,000, was next divided by this, giving $\frac{1}{40{,}000} \div \frac{1}{20}$ or $\frac{20}{40{,}000}$ or $\frac{1}{2000}$ as the ratio for the tidal period. The latter being 12.6 hours, or 45,360 seconds, the proportion for the model became $\frac{45{,}360}{2000}$ or about 23 seconds, and this figure was used in the experiments. The sand was spread horizontally when commencing the experiments, and after working for some time, conditions of channels and shoals developed in the surface which represented very closely the configuration of the estuary in 1834, although, owing to the proportionately small effect of the water on the sand, the full depths were not obtained. Many substances were tried to remedy this, such as powdered brick and pumice stone, flour of sulphur, sawdust, etc., but the most suitable material proved to be a very fine clean sand, slightly mixed with peat. The natural conditions having been thus reproduced, training walls of tin, of proper height and shape, were placed at the upper end of the estuary, representing the actual training walls then in existence. The changes in the model resulting from

this represented very closely, and in some points exactly, the changes which the walls had produced in the estuary itself.

Having succeeded so far, the operators introduced other training walls to represent the plans most favored by experienced engineers for continuing the improvement of the estuary. Fig. *a*, Pl. 41, shows the results of the first one after working the model for 6000 tides. The north wall was low; the south wall was at neap-tide level; and the dike across the island was above high tide, thus cutting off most of the inflow. This arrangement resulted in a fair depth between the walls, with a deep hole at their outlet; but owing to the reduced inflow and to the lack of direct connection between it and the ebb current, a shallow and tortuous channel gradually formed as shown, and an accumulation of sediment took place inside the estuary, seriously reducing the volume of the water. Sand-banks also formed on the seaward side of the dike and extended down the coast.

Fig. *b* gives the results of a second plan after being subjected to 3500 tides. This plan comprised a dike at the entrance as in the foregoing, and the prolongaation of the southern portion only of the existing works. The resulting channel followed in general the direction of the wall, but the depth of water was less regular than in the preceding experiment, owing to the open space to the north. The tortuous channel to the dike, the hole at the outlet of the double walls, and the advance of the foreshore reappeared again.

Fig. *c* shows a somewhat similar plan after 3700 tides, the difference being that the south wall was extended to the end of the dike. The channel became continuous along the wall, and other features were generally similar to the preceding case. The conditions above Honfleur, however, were not improved, owing to want of uniformity, apparent in the model, between the directions of the flood and ebb currents.

In Fig. *d* the walls were continued to the west, the southern one ending at Ratier Island. The result was a fair width and a tolerably uniform depth, the location of the channel being as shown. Deposit occurred behind the training walls on each side. This result was obtained with 6500 tides.

The channel of Fig. *e* was the result of 5000 tides. Close to the concave sides of the walls good depths were obtained, but shoals appeared in many places, and a large bar formed where the channel shifted from the south to the north wall. Considerable deposit took place behind the walls.

In order to ascertain how far results might be due to accidental arrangement of the bed, this experiment was tried again later after rearranging the sand in its natural topography in the estuary. The result is shown in Fig. *f*, and resembles fairly that of Fig. *e*, if allowance be made for the naturally shifting condition of channels in wide outlets.

Fig. *g* represents a modification of a design recommended by a commission.

The portion forming the last bend of the northern wall was kept low, while the others were made high. After 3700 tides, the resulting channel was found to be shallow in places, and though deep along the concave face of the southern wall, a shoal appeared just beyond. Outside the walls the main channel was of fair depth, but narrow and crooked, and shifting in location. The plan was then subjected to 6300 tides more, with practically similar results, the main channel still being unstable, as in the actual estuary. Behind the walls accretion took place.

Fig. *h* represents the plan recommended by theory, as facilitating the influx of the tide and avoiding any abrupt change of course and therefore of velocity. The southern wall was raised above high water, and the northern wall was gradually lowered from opposite Honfleur to Havre. After 7300 tides the trained channel was found to have a good width and fair depth, except at the shoals lying against the walls as shown, and was slowly improving. The depth in all the outlet channels was well maintained.

While it is of course impossible to reproduce all the forces at work in an estuary, such as the winds, waves, transportation of sediment, etc., the effects of the principal ones—the ebb and flow—can be fairly represented, as proved by these experiments and by those with a similar model of the Mersey, and later with one used by the engineers in charge of a project for improving the outlet of the Tiber.

Descriptions of a few leading examples, indicating the effects resulting from actual works, are given in the following paragraphs.

**Mississippi River, South Pass.**—(Pls. 36, 37 and 38.) The mouth of the Mississippi River, which drains nearly a million and a quarter square miles, is divided into three main outlets—Southwest Pass, South Pass, and Pass-a-l'Outre. As early as 1726 an improvement of the outlet was attempted by harrowing the bottom, and that and other means, such as dredging and partial jetties, were tried for many years with small success. Finally the construction of a canal at an estimated cost of ten million dollars was recommended, but the project was suspended by the proposal of James B. Eads in 1874 to construct at his own risk the present jetties. After numerous delays this proposal was accepted, and the jetties were built between 1875 and 1879.*

South Pass is about 12.9 miles long, with an average width of 750 feet, and a least original depth in the channel inside of 29 feet. The original depth on the bar was 8 feet, and the depth on the shoal at the head of the passes, 17 feet. The discharge per second at New Orleans in extreme high water has been given as 1,740,000 cubic feet, and the amount of solid matter carried in suspension at such periods as 2000 cubic feet per second. The range between high and low water there is about 21½ feet; at the head of South Pass it is about 2½ feet. The velocity at the latter point is 5 feet per second, and the fall per mile in the

* For a history of the improvements, see Annual Report, Chief of Engineers, U. S. Army, 1899, p. 1914.

pass, 2½ inches. This outlet before improvement was estimated to have carried about ten per cent of the discharge of the three passes; the remainder was divided almost equally between the other two main passes. In 1910 it carried 11.2 per cent. Its low-water discharge was about 25,000 cubic feet per second, and it carried to the sea about 22,000,000 cubic yards of sediment per annum.

The jetties were built by James B. Eads, who contracted with the United States Government to provide a channel 26 feet deep and not less than 200 feet in width, and with a center depth of 30 feet, and to maintain the same for twenty years for a total cost of eight million dollars. It is stated that this channel was maintained for the twenty years (1879–1899) with the exception of about 500 days. The jetties at the mouth were placed 1000 feet apart, considerably more than the width of the river above, but they were contracted later by inner jetties to a width of 650 feet and by spur dikes to a width of 600 feet. The head of the pass had also to be improved by jetties in order to secure a deeper channel, and mattress sills were placed across the entrances to Southwest Pass and Pass-a-l'Outre in order to prevent their enlargement and a consequent diversion of part of the flow from South Pass. Data as to these works will be found in the table on p. 333 and on Pls. 36 to 38. The conditions here are but slightly affected by the action of storms or sand-drift.

By 1910 there had been secured throughout the channel over the bar a least available depth of 31 feet; this deepening had been obtained almost entirely by scour, although dredging was used to some extent.

The disposition of the sediment by the river is worthy of notice. A comparison of the conditions from 1875 to 1903 (Pl. 36), indicates that the bar has advanced very little; that the river has maintained a deep channel to the open sea; and that the greater part of the vast amount of sediment brought down in 28 years has been deposited to the east and west of the channel and behind the jetties. The survey of 1910 shows the same general location of channel across the bar as the survey of 1903 (Pl. 38). The bank on the south side immediately opposite the ends of the jetties and the somewhat abrupt turn necessitated thereby are a source of some inconvenience to ships descending in any current, and not infrequently they go aground broadside before they can make change of course.

**Mississippi River, Southwest Pass.**—(Pls. 36 and 42.) As the demands of commerce for greater depths could not be met by further improvement of the South Pass, the opening of the Southwest Pass was begun, with the object of providing a channel not less than 1000 feet wide, and not less than 35 feet deep. This pass is 15 miles in length, and its discharge varies from about 40,000 cubic feet per second to over 400,000. A survey in 1898 showed that Southwest Pass carried 41 per cent of the discharge of the three passes; South Pass, 9 per cent; and Pass-a-l'Outre, 50 per cent. It is about 2650 feet wide at its head or inner

entrance, decreasing to a minimum of 1250 feet, and is 3600 feet wide at its mouth, and carries channel depths ranging up to 80 feet, and in places to 100 feet. The depth on the shoal at the head was from 31 to 33 feet, and on the bar, which lay about 4 miles from the mouth, the maximum depth was 10 feet. In the 24 years preceding 1898 the crest of the bar advanced on an average 176 feet per annum. Once the crest is passed, however, the water deepens rapidly, a depth of 50 feet being reached in a distance of 1500 feet. The maximum velocity across the bar was about 4 feet per second, and the average tide range 1.1 feet.*

The project contemplated securing a channel 35 feet deep at mean low water and 1000 feet wide. The method for obtaining this result consisted in dredging and in the construction of two jetties to maintain the channel when dredged. In addition to this work there was included the construction of sills across Cubit's Gap, The Jump, and Baptiste Collet's Canal, and the closing of all minor outlets between the forts.

The construction of the jetties, as originally contemplated, was completed in 1908 at a cost of $2,627,000, but it was found necessary to extend them later, the east one 3000 feet and the west one 3750 feet, owing to the fact that the bar had advanced seaward after the original survey had been made. This work was placed under contract in 1911, up to which time nearly 12,000,000 cubic yards of material had been dredged from the channel and the available depth in the outer 6.7 miles was 31 feet. In the main body of the pass the natural depth ranges from 40 to 80 feet, as above stated. On April 18, 1911, a steamer drawing 30½ feet, the deepest draft vessel which had sailed from New Orleans, passed through this channel to the sea. The use of the channel has been delayed in order to not interfere with the dredging, the South Pass affording an excellent outlet for present commerce. The work to June 30, 1911, had cost $5,620,000.

The jetty mattresses are 200 feet long, 2 feet thick, and vary in width from 35 feet for the surface or top mattresses, to 100 and 150 feet for the bottom or foundation mattresses. Some of these were built in place, on the mud, and loaded with 40 pounds of stone per square foot of mattress, while others were built on ways, launched and towed to position and sunk with 50 pounds of stone per square foot. When brought to the proper height the surface mattresses were covered with stone to a depth of 2 feet and a width of 18 feet, and a concrete cap with 12 feet of base and 8 feet of top width and a height of 4½ feet was put on.

During the construction some of the mattresses at the sea end of jetty shifted from their original position after having been in place several days, moving seaward about 100 feet. At the time of movement they had become covered with a layer of sand from 1 to 10 feet in thickness, giving evidence of a phenomenal movement of material along the bottom.

* Annual Report, Chief of Engineers, U. S. Army, 1900, p. 2289, and after.

The concrete capping was omitted from the inner 5600 feet of the east jetty and from the inner 3000 feet of the west jetty, as a rapid shoaling took place on both sides of both jetties. During the first year after placing the concrete caps an average settlement of the jetties of nine-tenths of a foot occurred, ranging from two-tenths to 12½ feet, but generally not exceeding 1.8 feet. The total average subsidence to June 30, 1911, was 1.6 feet for east and 1.8 feet for west jetties. There has been a deposit of sediment on the sea side of both jetties sufficient to reduce the depth of water to zero at mean low water over a considerable area, and there has also been a deposit along the channel side of both jetties out to station 11,000 on the east and to station 6000 on the west jetty.

**Columbia River (Oregon).** (Pl. 43.)*—This river offers an interesting example of single jetty work. It flows into the Pacific through a wide tidal estuary, which narrows to a width of 3 miles at the mouth. Its bed, the shoals, and the bar are composed of a fine sand, easily shifted. Little sediment, however, is brought down by the river. The mean tidal variation at the mouth is 7.4 feet, and the maximum 9.5 feet, the effect at extreme low water being noticeable for 150 miles from the coast. The tidal outflow is estimated as from 1,350,000 as the average to 3,000,000 cubic feet per second as the maximum. The estimated fresh water discharge is from 90,000 to a maximum of 1,500,000 cubic feet per second. The main channel current on the bar during the ebb runs at all seasons from southwest to west-southwest, with velocities from 2½ to 5½ miles per hour; the flood current runs from north to north-northwest, with velocities from 1½ to 3½ miles per hour. There is a littoral current running at its maximum from 2 to 3 miles per hour with a marked resultant set, due to prevailing influences, towards the north. The sand-drift is therefore northward also, and during the construction of the jetty the sand accumulated on the south side till it overtopped the work. There has been manifest at all times a noticeable tendency for the channel, or channels where two existed, to cross the bar on a southwest course. The depths on the bar have varied from 19 to 27 feet.

In the earliest existing chart of the entrance, made in 1792 by Admiral Vancouver, only one channel appears, running almost due west and carrying 27 feet over the bar. The next survey, made in 1839, shows two channels: a southerly one with a bar depth of 27 feet, and a northerly one with a corresponding depth of 19 feet. This condition remained a typical one for more than forty years, the principal changes being the gradual lengthening of Clatsop Spit, and the disappearance of the Middle Sands when the currents tended to reunite into a single channel. During this period the bar depths of the two channels varied between 19 and 27 feet. By 1885, however, the north channel had practically disappeared,

* Annual Report, Chief of Engineers, U. S. Army, 1903. The plate is reproduced from the Transactions Am. Soc. C. E., vol. liv, Part A, 1904, by courtesy of Lieut.-Col. Cassius E. Gillette, Corps of Engineers, U. S. Army.

since which time the south channel alone has been in existence, although about 1881 a minor channel opened still more to the south, which promised, until checked by the jetty, to create a second main channel.

During the construction of the jetty between 1885 and 1896, the channel swung northward, and in 1895 had a depth of 30 feet over a width of seven-eighths of a mile, and ran almost due west to the bar. This was the best condition attained. The northward trend continued, however, and by 1902 the depth had decreased to 22 feet, the remains of the old channel then pointing to the north, and two new channels of almost equal depth had become apparent. It is worthy of notice that during all the changes between 1885 and 1897 the channel across the bar pointed persistently to the southwest, and that when it swung to the northwest during 1897 and 1898 its deterioration commenced. This change of direction was due to the sand-drift from the south flowing round the end of the jetty, which ended in comparatively shallow water. The evidence shows that this drift is principally due to local movements of the sand, and that there has been no extension of the southwest face of the bar since 1839.

The south jetty, constructed between 1885 and 1896, was intended to secure 30 feet of water in the channel, in which object it was for some time successful. It was later proposed (1905) to obtain a depth of 40 feet with a width of not less than one-half mile. For this purpose the south jetty was to be extended 2½ miles to deep water, and to be raised to mean tide level, and should this fail to secure and maintain the desired channel, a north jetty 2½ miles long or less was to be built, as shown on Pl. 43. This would locate the outlet on a portion of the bar that had remained practically unchanged since 1839. To expedite the action of the water, suction dredging was commenced and has continued steadily.

The north jetty was not commenced, however, when proposed, and in 1910 further recommendation was made for its construction, with a view to obtaining a deeper and more permanent crossing than the single jetty appeared able to secure. At that time there was a channel over the bar with a width of 8000 feet and a least depth of 24 feet, its center portion having a least depth of 26½ feet with a width of 1000 feet.

Conditions at this outlet are unusually difficult, as the coast is exposed to very heavy seas, and there is a considerable sand-drift working north.

**Danube.*** (Pl. 39.)—The Danube, which is about 2000 miles in length, drains 300,000 square miles, and carries out more than a million cubic feet of water per second and some 600,000 cubic yards of sediment in twenty-four hours, during floods. The Sulina mouth, which was selected for improvement largely owing to restricted funds, discharges in floods about 75,000 cubic feet per second. There is no tide, although the range of level is more or less affected by the winds,

* Proceedings, Inst. C. E., vol. xxxvi.

which usually blow from the north and east. A littoral current running from ½ to 1 knot per hour from north to south is frequently in evidence.

Temporary works, consisting of two jetties each of three rows of piles, were begun in 1858. The outside row was driven close, and riprap, consisting of pieces each less than a cubic foot, was thrown on each side till it reached the water line. During the succeeding five years the storms persistently beat down this stone to a level of 3 to 4 feet below the water, beyond which depth, however, no effect was produced, except near the pier heads, where the stone was beaten down to a depth of 14 feet. The sea slope and the inner slope of the riprap took the final inclinations of 2½ horizontal to 1 vertical and 1½ to 1 respectively. These jetties resulted in an increase of depth on the bar of between 6 and 8 feet. Between 1867 and 1871 they were changed into permanent structures by the addition of concrete tops, partly built in place and partly composed of blocks already hard when set in position, with concrete flushed around them. The original depth on the bar was from 7 to 11 feet, and in 1872 it had increased to between 17 and 20 feet, although liable to variation from deposit of sediment. This deposit, however, was usually removed by storms. It was found that the crest of the bar had advanced but little and that extensive shoaling had taken place under the shelter of the south jetty, and a corresponding erosion along the north jetty. Observations taken during the ten years after 1871 showed that the rate of advance of the bar had been reduced by more than one-half. The shoaling under the southern jetty is said to be due to the eddy of the southeasterly littoral current; the erosion along the north jetty to the action of the waves, which was found to be much more violent after the jetty had been consolidated. This change of coast line is very noticeable on the map for 1900. (Pl. 39.)

The financial results of these works was to reduce the expenses of shipping in passing the mouth to one-seventh of the charges previously in force.

By 1890 the increasing draft of vessels found the limiting depth of 20 ft. insufficient, and accordingly in 1894 a pair of inner jetties was built as shown on the map, and dredging commenced, with the result that the depth available in 1895 was 24 feet. This depth has since been maintained, with slight variations, from year to year, the quantity of material dredged between 1895 and 1900 varying from about 270,000 to 430,000 cubic yards per annum.* The cost, including all expenses except interest, depreciation, and insurance, was about 8½ cents per yard.

**St. John's River.**† (Pl. 40.)—This river, whose watershed of some 7500 square miles occupies the greater part of the peninsula of Florida, opens into the Atlantic through a comparatively narrow mouth (about 1700 feet in width), although the

* Proceedings International Congress of Navigation, Glasgow, 1900.

† Annual Reports, Chief of Engineers, U. S. A., 1879, and others.

estuary widens considerably inside. At Jacksonville, 25 miles above the outlet, the river is from 2000 to 2500 feet wide, with depths as great as 66 feet. Observations made in 1878, before any permanent improvement was attempted, gave an average tidal inflow of 1,181,200,000 cubic feet per tide, and an average outflow of 1,860,300,000 feet per tide. The distance from the mouth to the bar was 2½ miles, with a depth on the bar of 5 to 7 feet, and a tidal range of 4 to 6 feet. The river carries but little sediment, and the bar is formed by the action of the waves and sand-drift, the resultant tendency of which, along all the southern part of the Atlantic coast, is noticeable towards the south. The entrances in this zone usually consist of two channels which shoal and deepen alternately, one opening about northeast, the other about southeast, their depth and direction varying somewhat with the winds. The southeasterly channels appear to be the more persistent in holding their location.

At the mouth of the St. John's similar conditions were present, and in 1879, after dredging had been attempted, it was decided to build two jetties with the object of obtaining a bar depth of 15 feet. At that time the best depth was found in the northerly channel, the southeast one having shoaled, and it was accordingly chosen for improvement. The general trend of the outlet channels, according to records available up to that time, is stated to have been at right angles to the coast. By the plan chosen the north jetty was to be 9400 feet long and the south one 6800 feet. The work, however, was always hampered by scanty funds, and in 1894 it was decided to extend the jetties and to attempt to open a low-water channel 300 feet wide and 24 feet deep by dredging. The report for 1903 stated that the available depth on the bar was then 14 feet, while the bar itself had advanced seaward 350 feet in the preceding 12 months. The channel dredged across it during the winter had filled up, and there appeared to be more or less shifting of the depths and widths inside. By 1910, however, there had been secured an available depth of nearly 24 feet. The survey that year, compared with that of 1903, showed practically the same alignment of the channel. The shoal inside the north jetty, adjoining the squares marked 3 and 4 (Pl. 40), had filled up considerably, and south of the south jetty some shoaling had also taken place. The peak of the 36 ft. contour opposite the entrance had moved outwards about 1500 feet, but little or no advance had taken place during the preceding twelve months. While a good deal of sand had been washed over the north jetty during storms, the current appeared to have carried it out to sea.

**Panuco.** (Pl. 44.)—This river, which forms the harbor of the Mexican port of Tampico, is largely dependent on its floods for the maintenance of necessary depths. The channel is carried to deep water by two parallel jetties, completed in 1892 and about 1000 feet apart. The tide range is a little over one foot.

The floods in this river are very variable, an interval of three to five years sometimes passing between them, but their violence apparently compensates for their rare occurrence. In 1893 a high rise came, and, confined between the jetties, scoured out over a million cubic yards from the harbor, creating a deep channel which the river has since maintained, the depth being 28 feet, and the width 580 feet.*

**Humber.**—The Humber, on the east coast of England, discharges through a wide and deep tidal estuary, and is an instance of a river whose outlet has needed no improvement. It drains about 10,500 square miles, its ordinary fresh-water flow being about one-eightieth, and its flow in floods about one-eighth of the tidal flow. In spring tides the tidal flow amounts to 2,200,000 cubic feet per second. A large amount of detritus, at times amounting to 320 grains per gallon, is brought down. Although there is a considerable amount of shingle traveling southward during gales, a low-water channel more than 70 feet deep and nearly 3 miles wide, has always existed between the mouth of the estuary and the open sea. The entrance is protected on the north by Spurn Point, and the only works of improvement needed have been those built to arrest the erosion of this headland.† A large part—nearly three-fourths—of the estuary has been reclaimed in the course of time.

**Liffey.** (Dublin Harbor.)—This outlet is protected by two walls. The south wall was completed in 1796, being about 3 miles long, and was built above the level of high water. Its effect was to protect the river from the northerly drift of the sands, and some deepening of the bar resulted. The north wall was not finished till 1819, and was made 9000 feet long. The walls are 1000 feet apart at the entrance and inclose a tidal basin of 2350 acres. For about one-half of its length next the shore the north wall was built to 6 feet above high water, and then dropped to the level of high water (for lack of funds), and thence sloped gradually down to below low water. Before the building of this wall the low-water depth on the bar was 6 feet, and the high-water depth 19 feet. In the first nine years the bar scoured out 3 feet, the total gain to 1873 being 9½ feet. Since then no deepening by the natural forces has occurred. The scouring action appeared to have ceased at about a mile from the bar, where the same depth existed as in 1819, namely, 16 feet. It is estimated that 960,000,000 cubic feet of water passes in and out with each average tide. The harbor and the outlet have been deepened considerably by dredging during recent years.

**Clyde.**—The outlet of the Clyde is a tidal estuary, well protected by natural conditions from disturbances of storms and littoral currents, and until the advent of deep-draft vessels, but little work was needed to maintain the improvements. Prior to 1773, the river was fordable on foot at Dumbuck Ford, more than twelve

* Transactions Am. Soc. C. E., vol. xlii. The cuts are reproduced by courtesy of Dr. E. L. Corthell.
† Minutes of Proceedings, Inst. C. E., vol. c., 1890.

miles below Glasgow, but in that year a limited number of spur dikes were constructed and their effects supplemented by harrowing and plowing of the hard shoals. These works deepened the channel at Dumbuck Ford, chiefly by scour, to 14 feet at low water in eight years. Some thirty years later many additional dikes were built, and the old ones remodeled and consolidated into parallel walls so as to give a channel 180 feet wide at Glasgow, with a gradual enlargement to about 700 feet at Dumbarton Castle. After that period the work of improvement by training dikes and dredging was carried on almost continuously, the plans being changed at various periods to suit the growing needs of commerce. Between 1844 and 1900 about 57,000,000 cubic yards were dredged from the river, harbor and docks, the material being at first placed to fill up low lands, and later taken to deep water and dumped. The effects of the works were to increase the range of spring tides at Glasgow, by about 2 feet between 1834 and 1880, and to lower the low-water level there 8 feet between 1758 and 1880. The greatest draft of vessels using the channel was 13½ feet in 1821, and 26½ feet in 1900. The channel requires continuous excavation in order to maintain the depth, as much silt comes in from the tributaries and much from sewage, the total amount per annum between 1890 and 1900 being estimated at 871,000 cubic yards.*

**Seine.** (Pl. 45.)—The estuary of the Seine before 1848 was unimproved and encumbered by sand-banks and shallow channels, so that boats of only 200 tons burden had much difficulty in navigation, and the journey from the sea to Rouen, a distance of 74 miles, required four days for a vessel drawing 10 feet. Between 1848 and 1850 rubble training dikes were built on each side of the upper end of the estuary, extending as far down as Quillebeuf, and as these gave good results, increasing by 10 feet the depths at certain points, the southern one was extended later to La Roque. This portion of the river, however, being uncontrolled by the single bank, failed to improve, and in 1861–62 the northern bank was built. The increase in width between these banks is 1 foot in 200. Many years later, in 1895, the dikes were consolidated and extended and systematic dredging was begun

The results of these works have been very satisfactory. The channel affords a least depth of about 16 feet at high tide, while the journey to Rouen now occupies only one or two tides. There has, however, been a very great and unexpected increase in the deposit of silt in the estuary since the first dikes were completed, but much of this has been reclaimed and was reported in 1904 to have a valuation of $4,500,000. The increase in public wealth due to these improvements, and without including the benefits to navigation, was estimated at 50 per cent more than their cost.†

* Proceedings International Engineering Congress, Glasgow, 1901.
† "Travaux d'amélioration de la Seine."

**Volga.***—The Volga discharges into the Caspian Sea through a delta containing 5300 square miles, and divides in its passage into a vast network of small channels. A great quantity of detritus is discharged, and it is claimed by some that the delta is advancing seaward at a rate of 1200 feet per annum; this, however, is considered by others to be an exaggerated figure. The first attempt at improvement was begun in 1858 in the Kamysiak branch, which then had a least depth of 8 feet, except on the bar, where the depth was only 1 foot below mean water. The crest of the bar was $1\frac{1}{3}$ miles from shore, and the depth beyond increased very slowly, being only 8 to 9 feet at a distance of 17 miles from the mouth. The project was to train and dredge the river so as to obtain a depth of 10 feet, but after an expenditure of $827,000, the works were abandoned in 1869, as it had become evident that they would not fulfill expectations. Since that time navigation has been assisted by continuous dredging. Owing to the extremely flat sea slope, any improvement by jetties would be enormously expensive, especially as there is no strong littoral current, nor any great depths in which the sediment would be disposed of. Similarly a lateral canal would be of too great cost, as its length would have to be about 125 miles. It appears probable, therefore, that the method of dredging will be retained.

Prior to the increase of depth thus obtained, loaded vessels with a draft of more than 8 feet had to lighter much of their cargo when entering or leaving the Volga, and two stations were established for this purpose, one known as the 9-foot roadstead, 15 miles off shore, and the other as the 12-ft. roadstead, 15 miles still further out. Hundreds of vessels lay there at a time without shelter, and long interruptions often occurred in the work, owing to storms. The expenses of these transfers of cargo were estimated at fifty million dollars a year, the number of men employed being at times as great as 10,000.

* Report on the Volga, by Major F. A. Mahan, U. S. A., 1904.

DATA REGARDING JETTIES.

| Locality. | Mississippi River. South Pass. | Mississippi River. Southwest Pass. | Danube, Sulina Mouth. | St. John's River, Florida. | Columbia River, Oregon. (a) | Columbia River, Oregon. (b) | Galveston Harbor, Texas. | Aransas Pass, Texas. | Harbor of Venice. Lido Entrance. | Harbor of Venice. Malamocco Entrance. |
|---|---|---|---|---|---|---|---|---|---|---|
| Single or double jetties | Double, parallel. | Double, converging at mouth. | Double, converging | Double converging, parallel at mouth. | Single | Double, converging (proposed) | Double, converging, parallel at mouth. | Double, generally parallel (k) | Double | Double |
| Begun | 1875 | 1903 | 1858 | 1880 | 1885 | ........ | 1888 | 1895 | 1881 | 1839 |
| Completed | 1879 | ........ | 1871 | 1905 (about) | 1896 | ........ | 1898 | 1912 (about) | 1907 (about) | 1872 |
| Original depth on bar | 8 ft. | 10 ft. | 7 to 11 ft. | 5 to 7 ft. | 19 to 27 ft. | 22 ft. | 12 ft. | 8 to 9 ft. | | |
| Depth on completion of jetties | 26 to 30 ft. | 35 ft. projected. | 19 ft. | ........ | 30 ft. | 40 ft. projected. | 25½ ft. | ........ | ........ | |
| Depth in 1911 | 29 ft. min. | 31 ft. | 24 ft. (dredged) | 24 ft. | 25 to 27½ ft. | 25 to 27½ ft. | 34½ ft. | 20 ft. | 24 ft. min. | 31 ft. min. |
| Normal tide range | 1.1 ft. | 1.1 ft. | None. | 5.0 ft. | 7.4 ft. | 7 4 ft. | 1.2 ft. | 1.2 ft. | None. | None, |
| Average length of portion between jetties | 2½ miles at S. pass, ¾ mi. at head of passes. | 2 miles. | 1 mile. | 2½ miles. | ........ | 4¾ miles. | 5 miles. | 1.5 miles. | 2.1 miles. | 1 mile. |
| Total length of jetties | 20,000 ft. at S. pass. | ........ | N. jetty 5332 ft., S. jetty 3457 ft. (in 1871) | N. jetty 11,934 ft., S. jetty 10,712 ft. (in 1903) | 22,440 ft. (on south side) | N. jetty 13,200 ft., S. jetty 35,640 ft. | N. jetty 25,907 ft., S. jetty 35,603 ft. | 16,150 ft. | N. jetty 11,800 ft., S. jetty 10,100 ft. | N. jetty 6896 ft., S. jetty 3108 ft. |
| Distance apart at outer ends | 600 ft. | 3000 ft. | 600 ft. | 1600 ft. | ........ | ........ | 7000 ft. | 1300 ft. | ........ | ........ |
| Materials | Mattresses, cribs, riprap. | Mattresses, riprap, concrete. | Concrete on riprap. | Riprap on mattresses. | Riprap on mattresses. | Riprap only. | Riprap, partly on mattresses. | Riprap, partly on mattresses, | Concrete on riprap. | Masonry on riprap. |
| Total cost of improvement | $8,000,000 (c) | (d) | $895,000 | $3,500,000 (e) | $1,960,000 | $5,425,000 (g) | $6,750,000 (h) | $2,280,000 | About $1,500,000 | $1,600,000 |
| Average cost per lineal foot of portion improved | ........ | ........ | $170 | $310 | ........ | ........ | $321 (j) | $285 (k) | $140 | $320 |
| Average cost per lineal foot of each jetty | ........ | ........ | $102 | ........ | $87 | N. jetty $91, S. jetty $216 | | | $68 | $160 |
| Annual cost of maintenance | ........ | (d) | ........ | (f) | ........ | ........ | ........ | ........ | ........ | $3200 |

in water 4 feet deep to about $110 in water from 7 to 11 feet deep.

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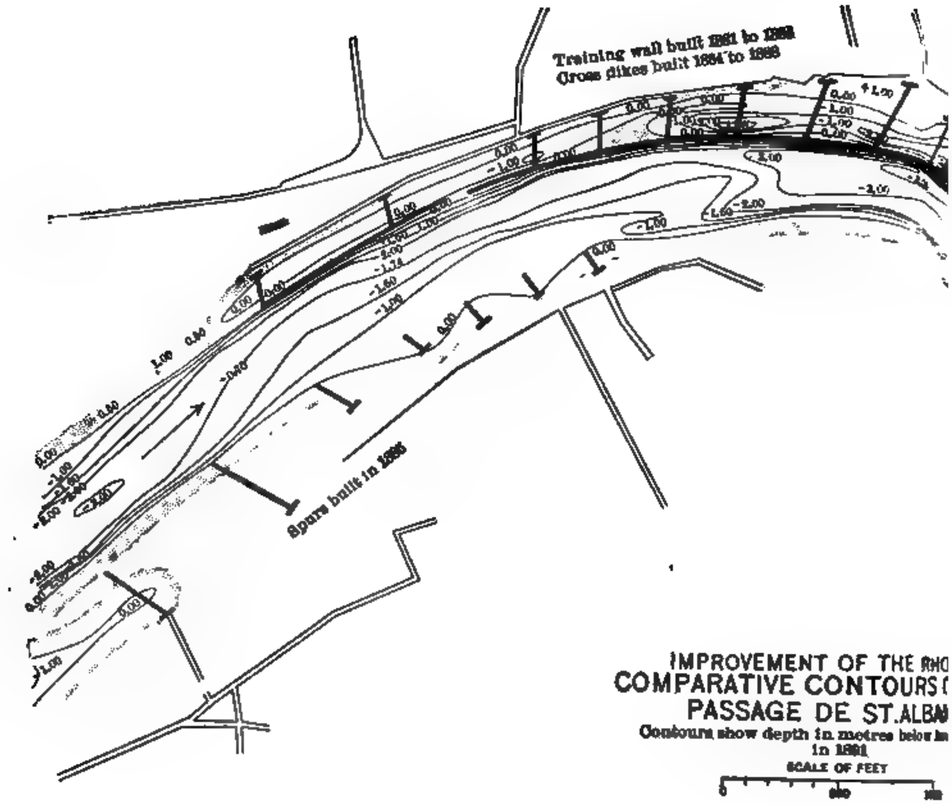
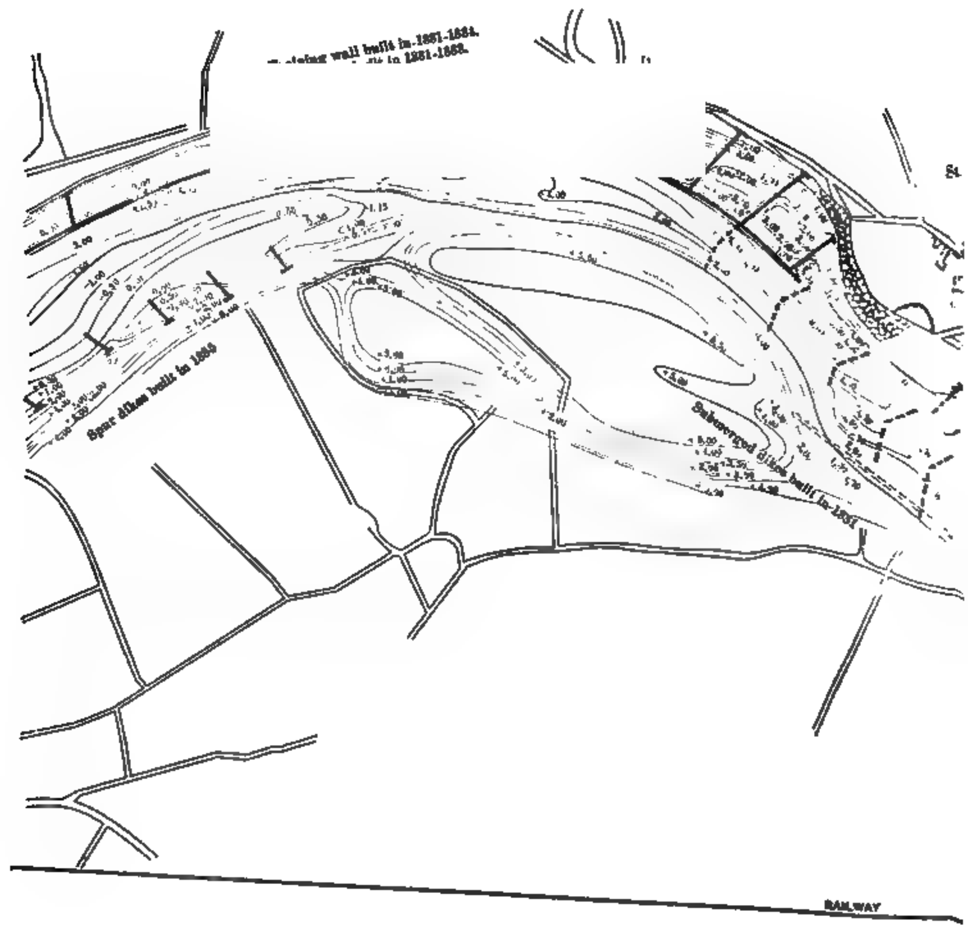
PLATE 1*a*

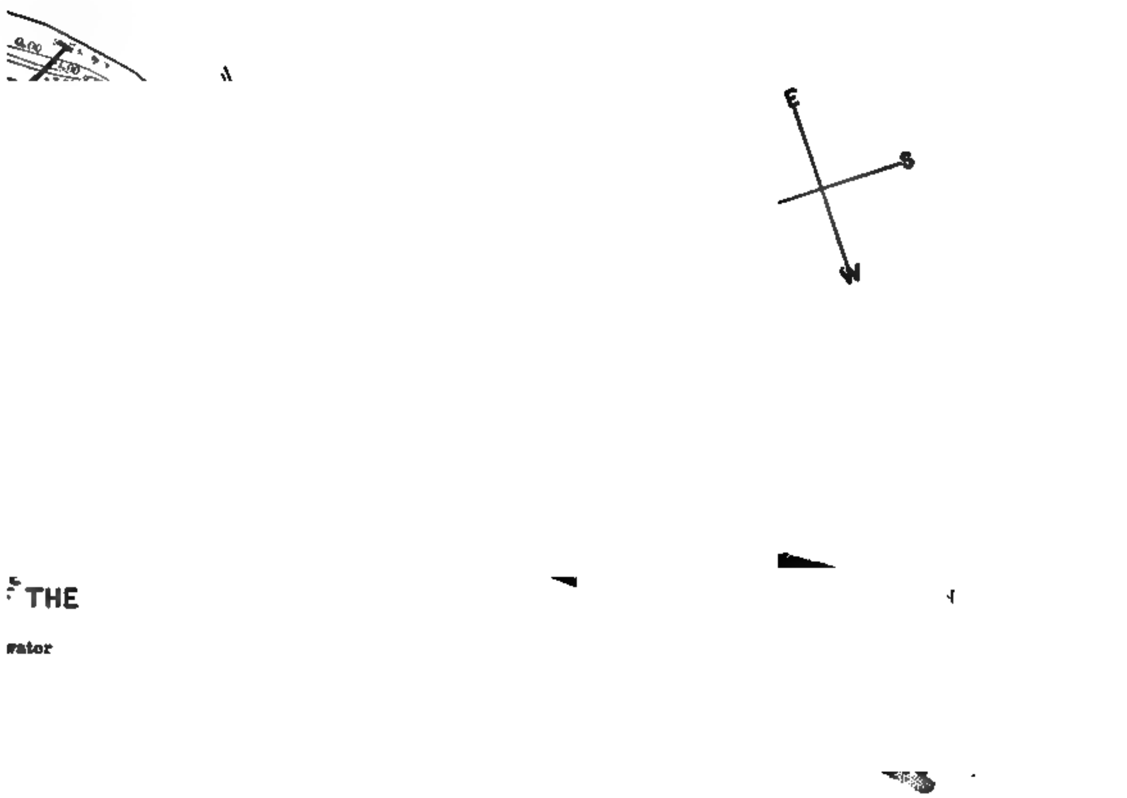
PLATE 1




IMPROVEMENT OF THE RHO
COMPARATIVE CONTOURS
PASSAGE DE ST.ALBA
Contours show depth in metres below lo
in 1881
SCALE OF FEET




IMPROVE
COMPARA
PAS
Contours show dep




10 meters below low water in 18[illegible].

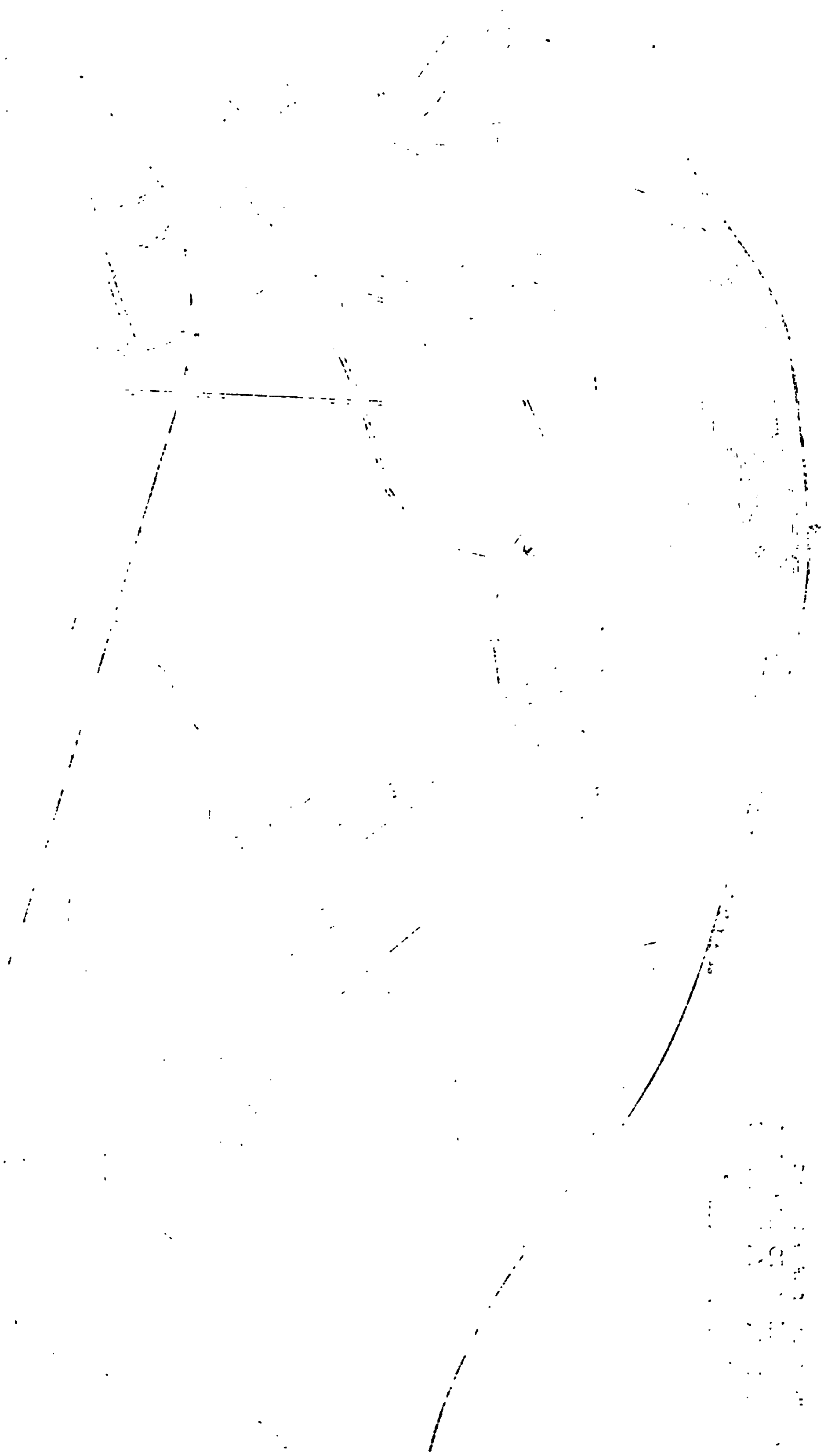
SCALE OF FEET

500 1000

Pl. 1.

(Reference, p. 77.)

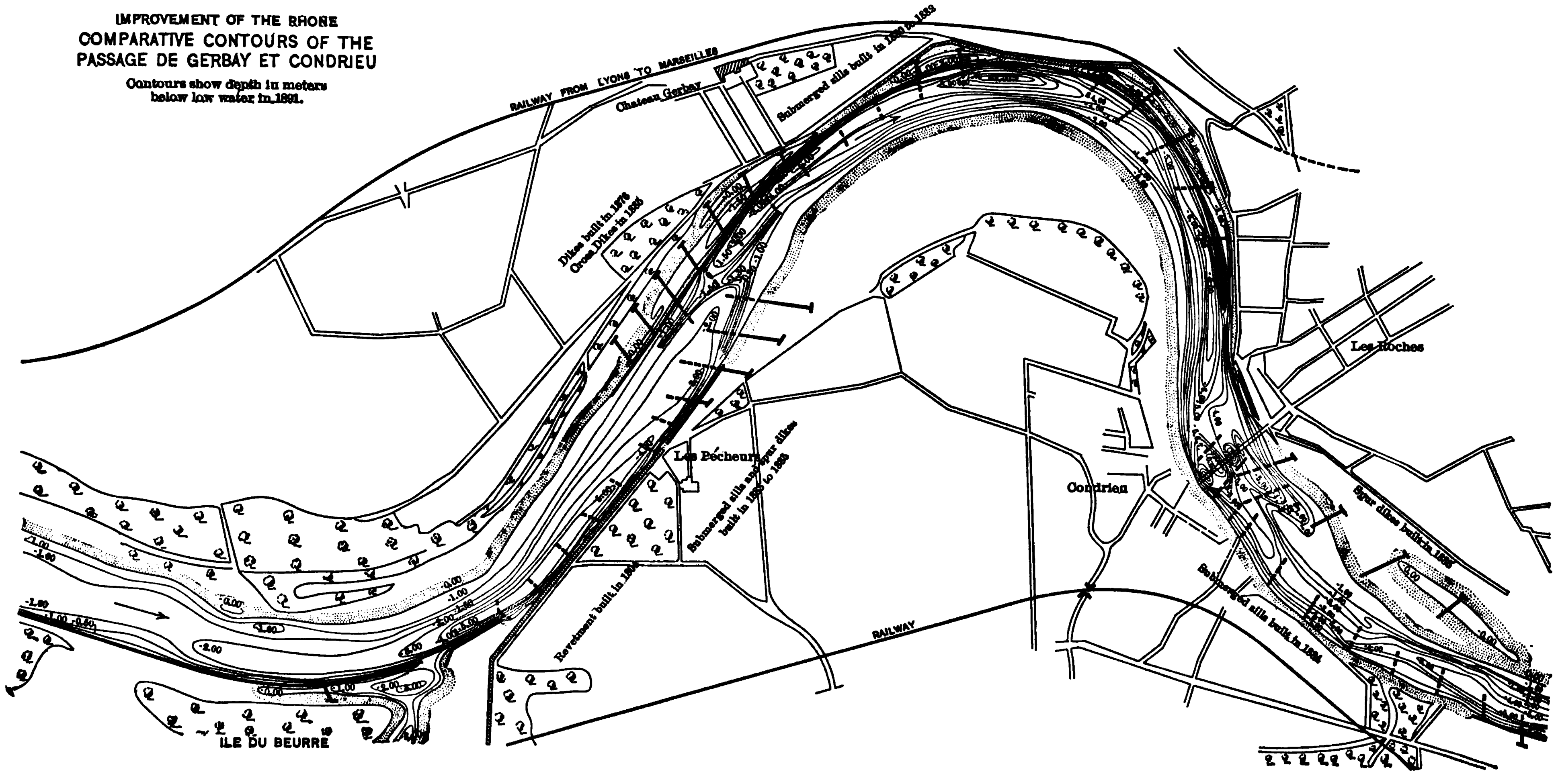
PLATE 2



IMPROVEMENT OF THE RHONE

COMPARATIVE CONTOURS OF THE PASSAGE DE GERBAY ET CONDRIEU

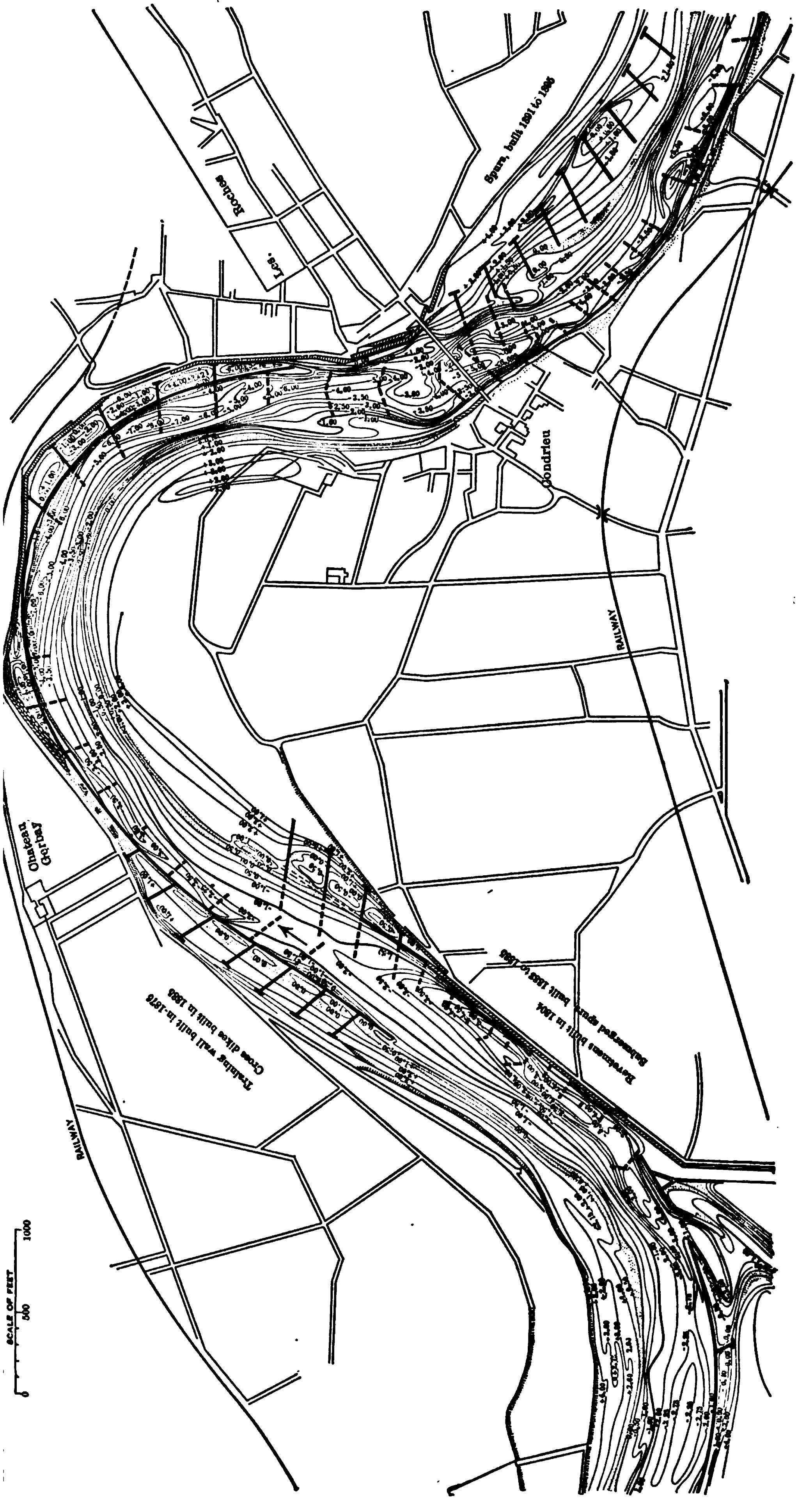
Contours show depth in meters below low water in 1891.




IMPROVEMENT OF THE RHONE

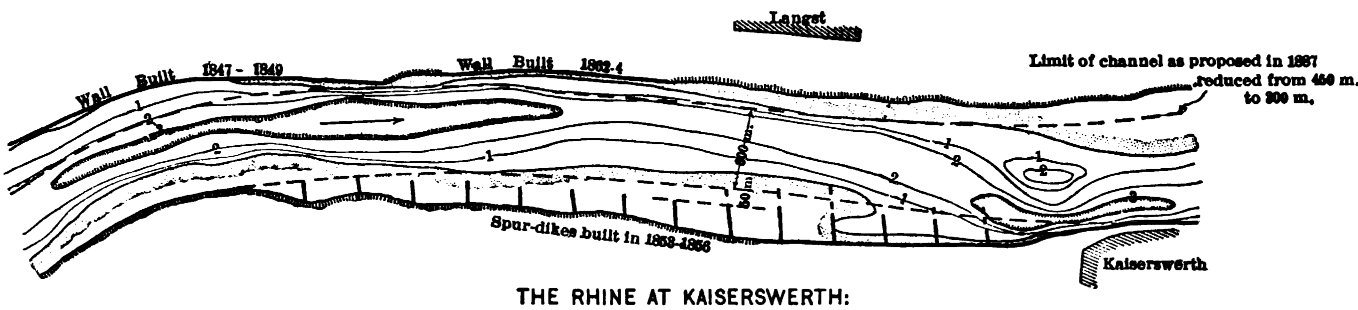
COMPARATIVE CONTOURS OF THE PASSAGE DE GERBAY ET CONDRIEU

Training wall and cross dikes built 1880 to 1884




**PL. 2.**

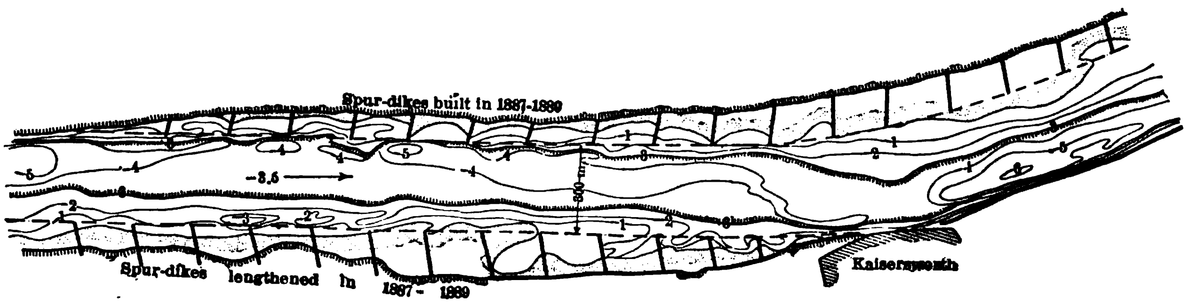
(Reference, p. 77.)




THE RHINE AT KAISERSWERTH:
CONDITION IN 1874

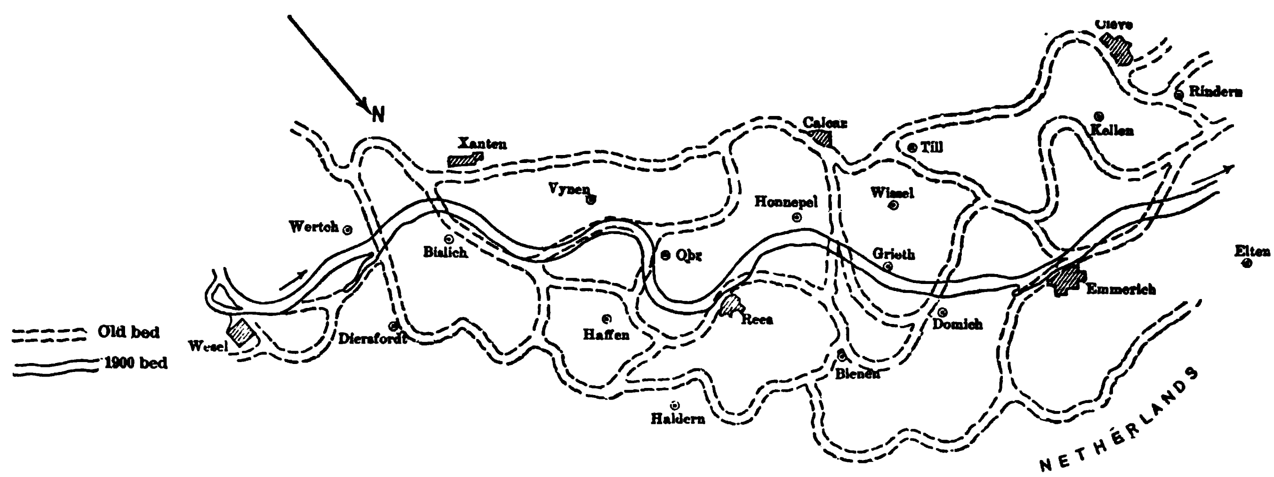
Contours show depths in meters below low water

0 200 400 600 800 METERS




0 200 400 600 800 METERS

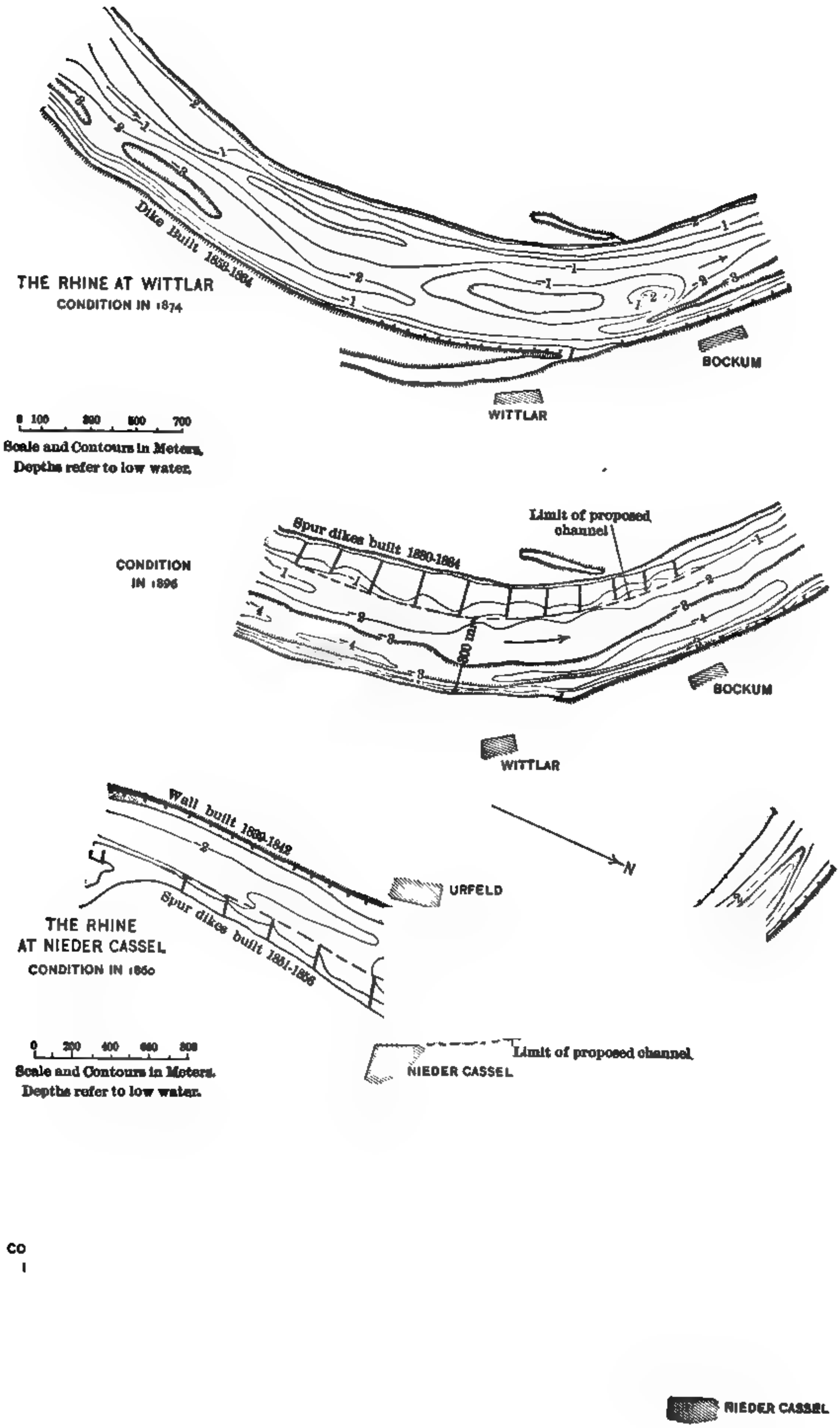
THE RHINE AT KAISERSWERTH:
CONDITION IN 1896




0 5 10 KILOMETRES

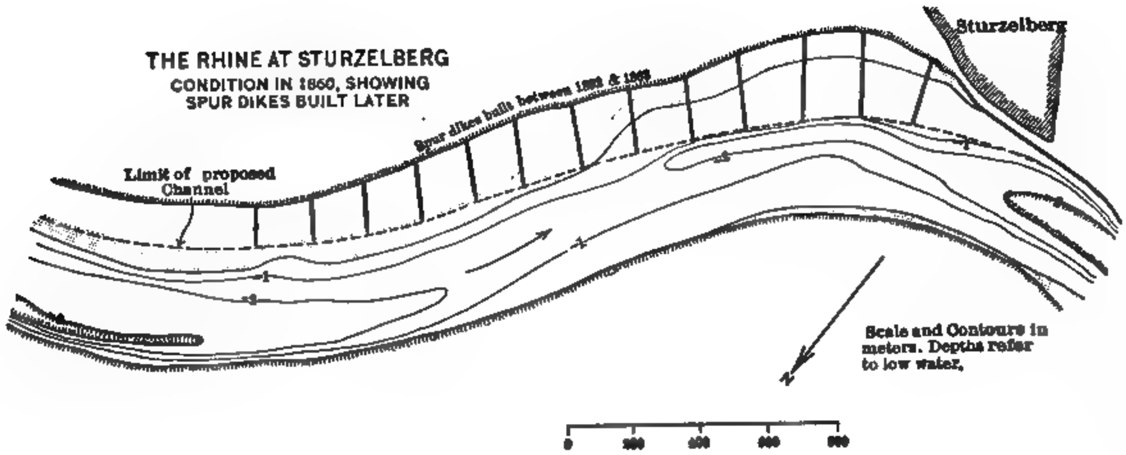
MAP SHOWING THE OLD BED AND THE 1900 BED OF THE RHINE,
FROM WESEL TO ELTEN

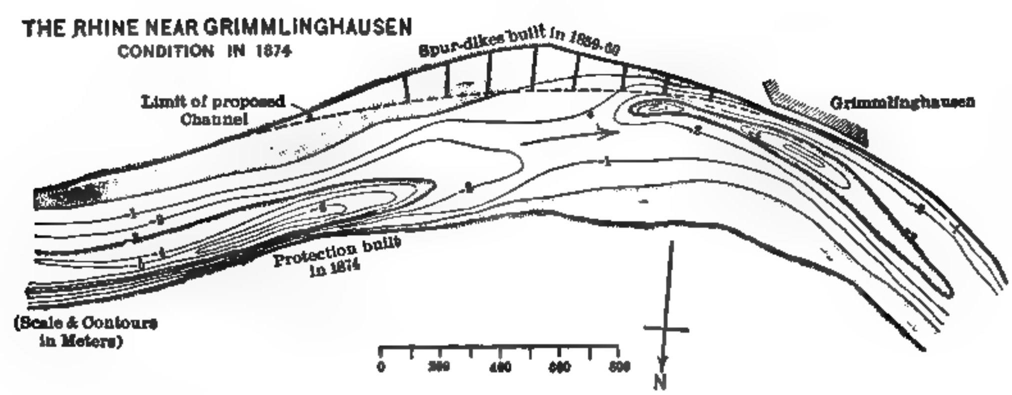
**Pl. 3.**
(Reference, p. 78 and after.)




PL. 4.
(Reference, p. 80.)

PLATE 5

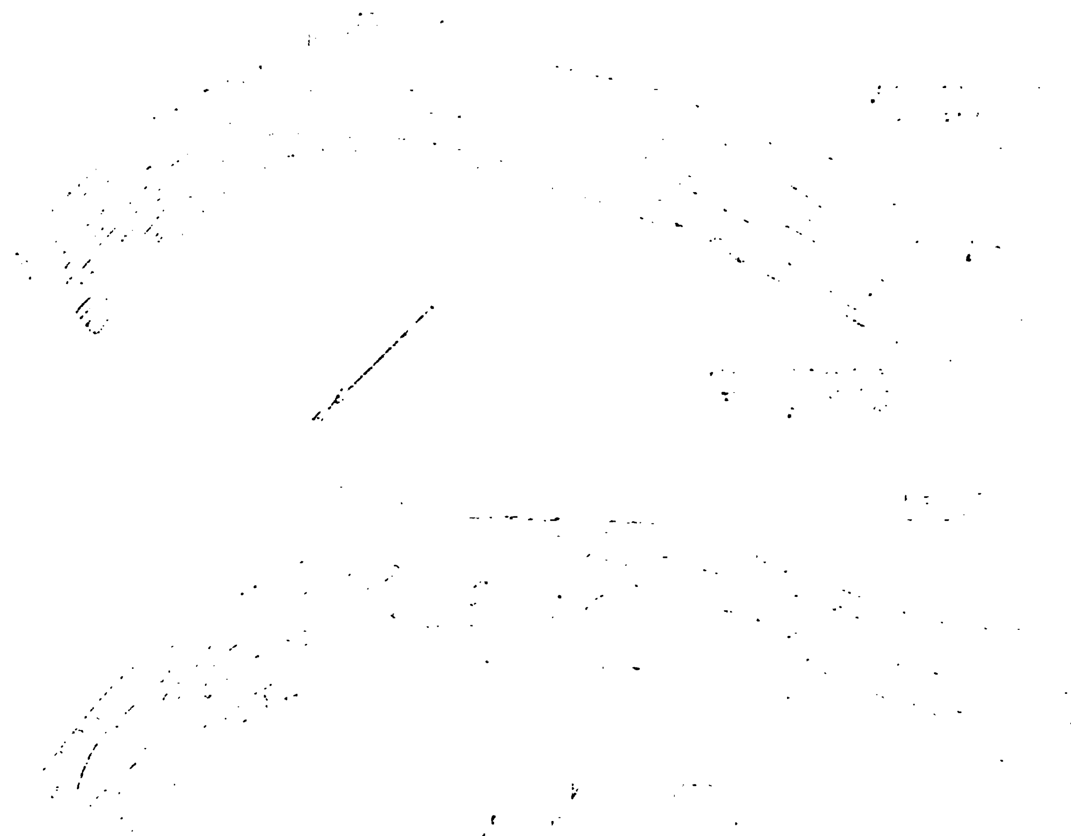


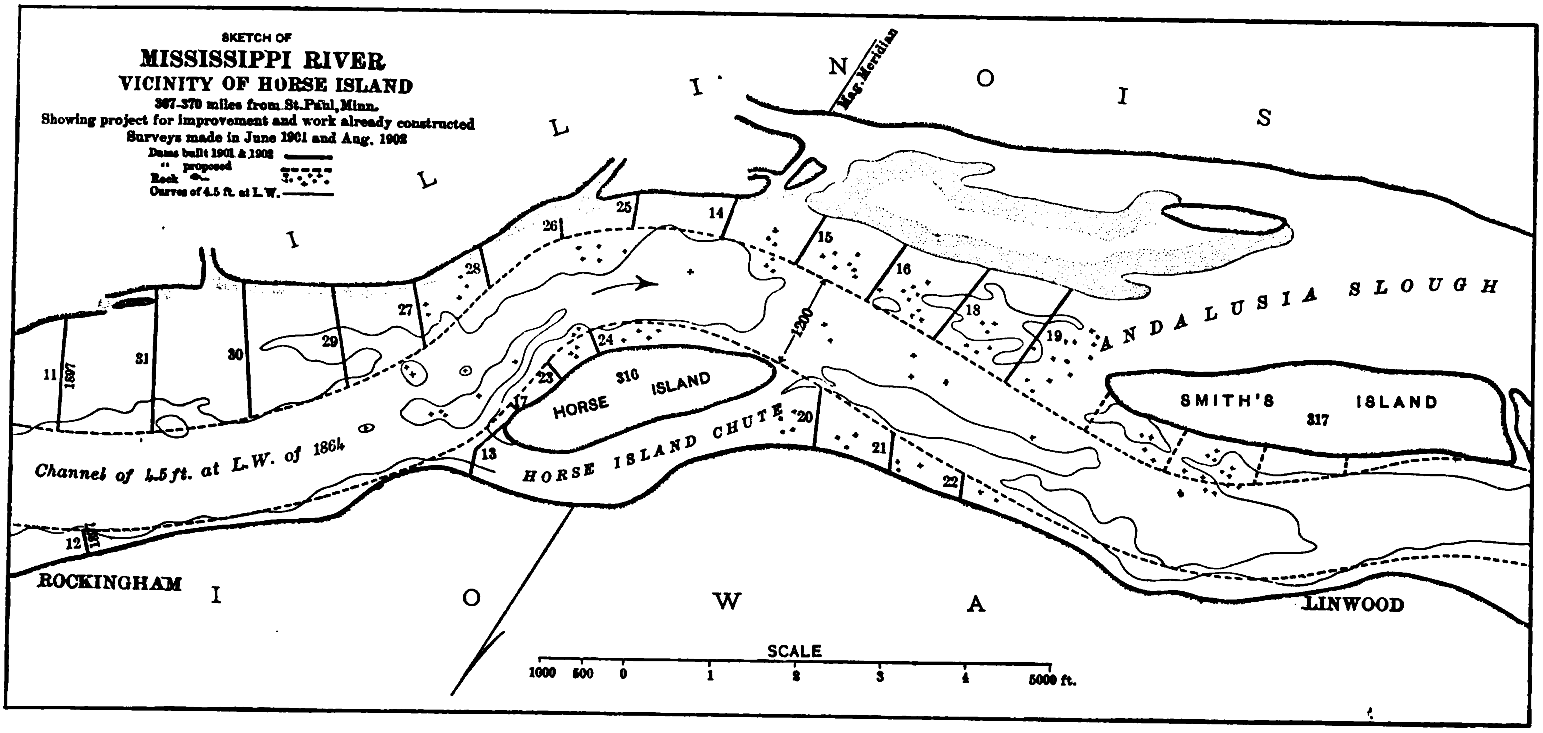



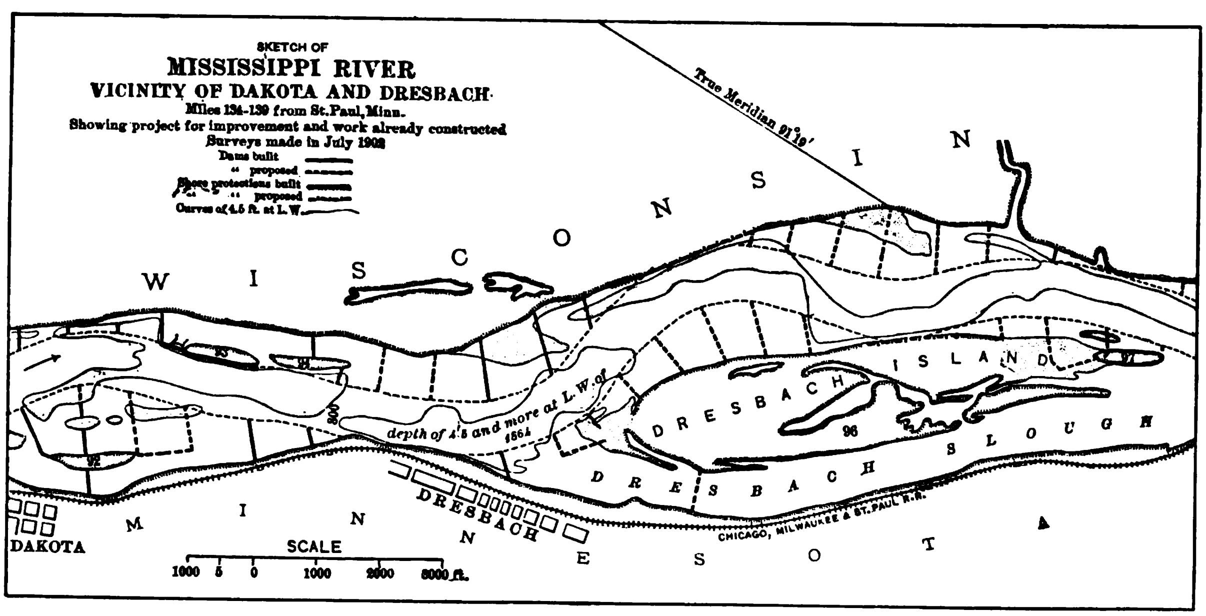

Secondary Spurs


Pl. 5.

(Reference, p. 80.)

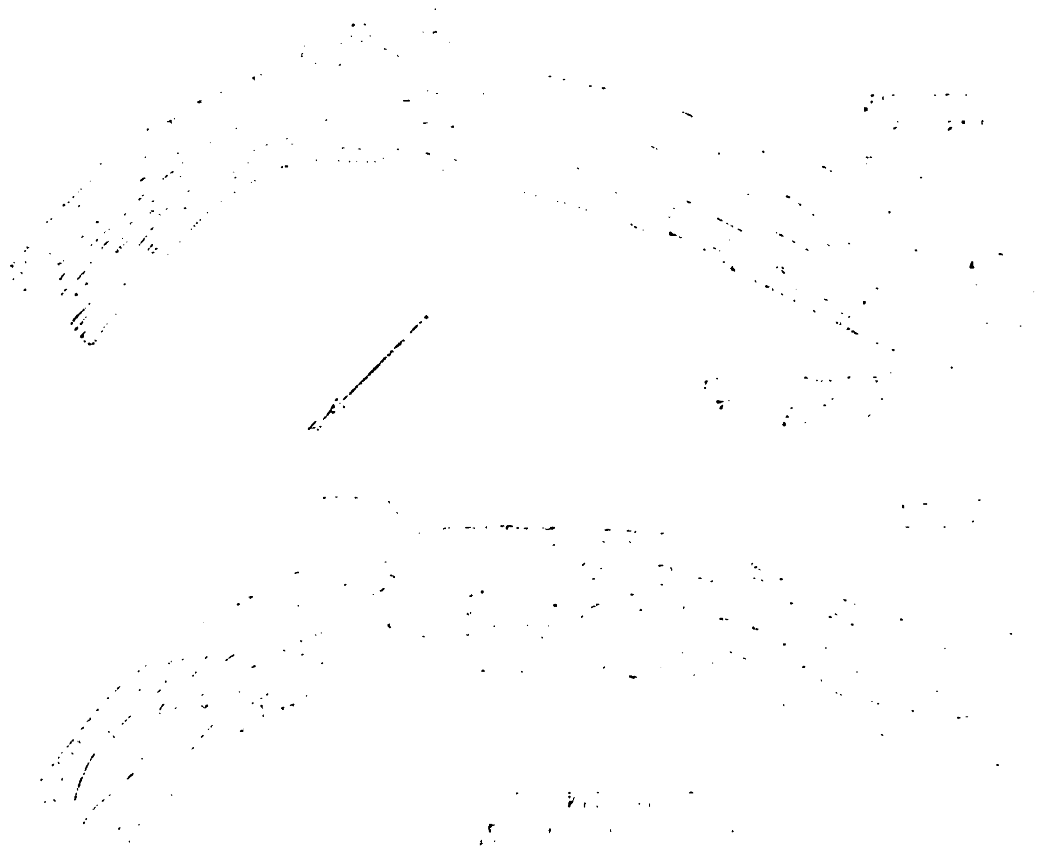


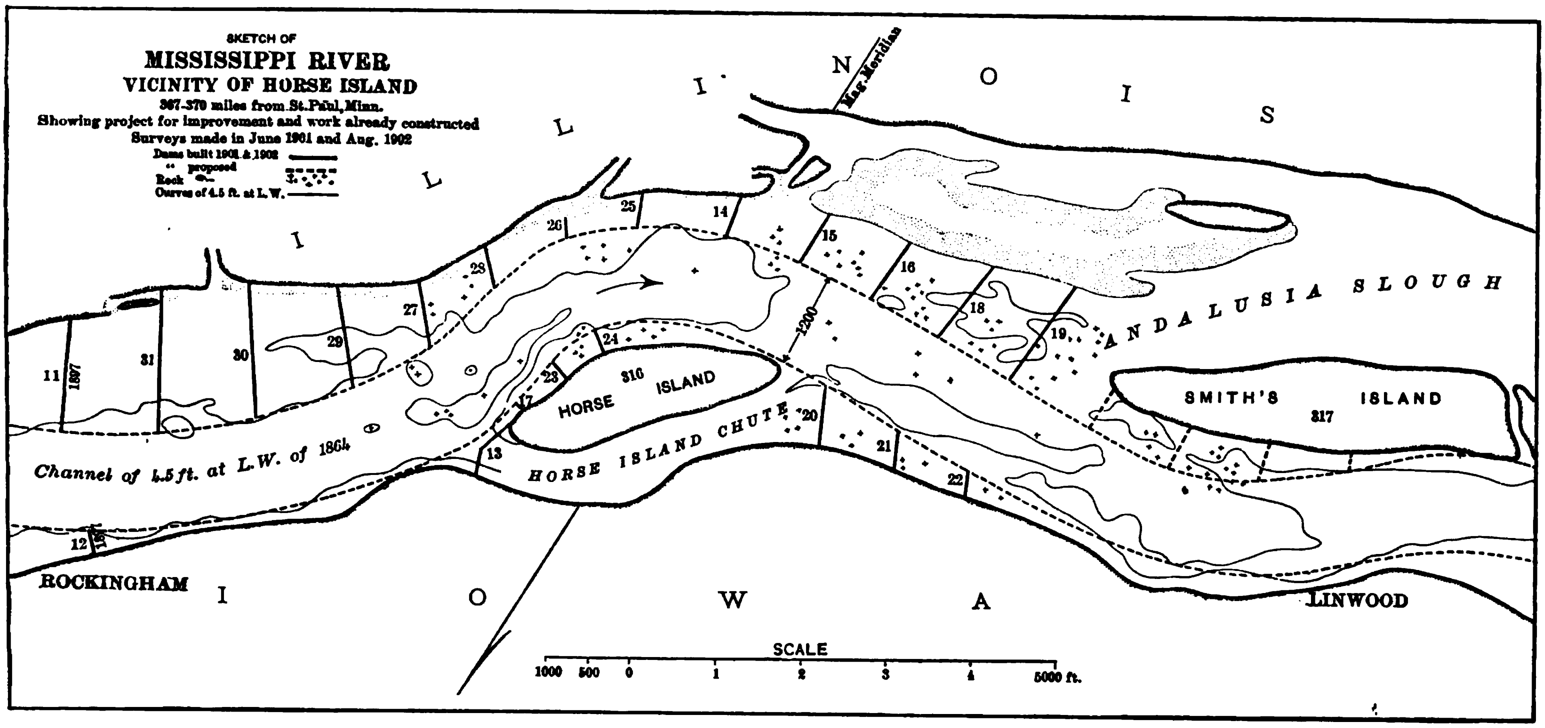


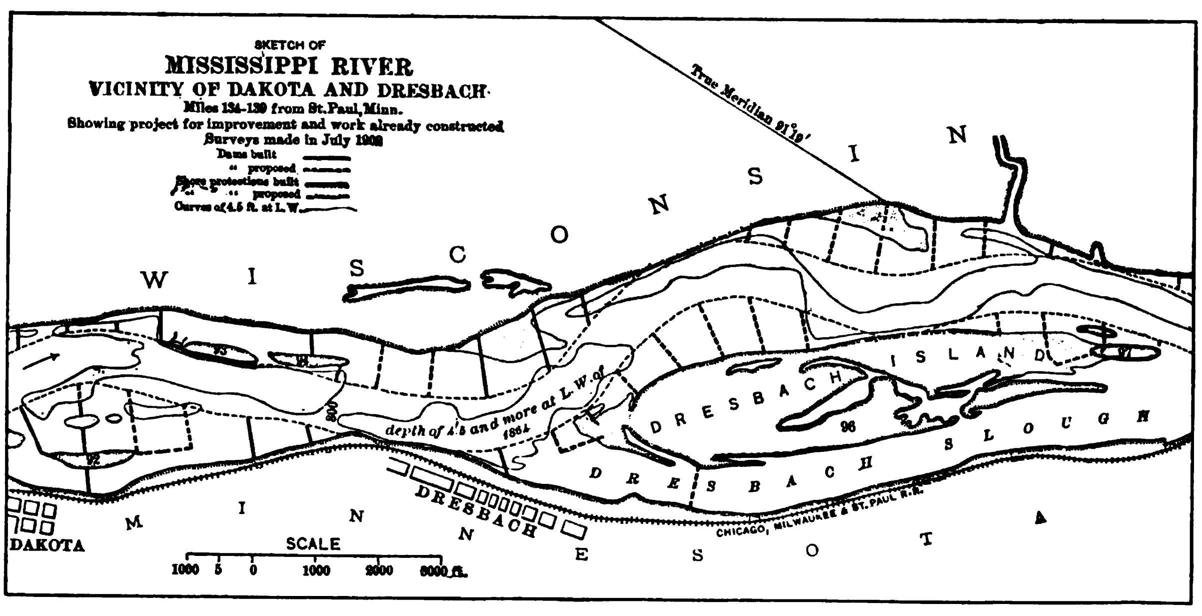




**Pl. 6.**

(Reference, p. 87.)



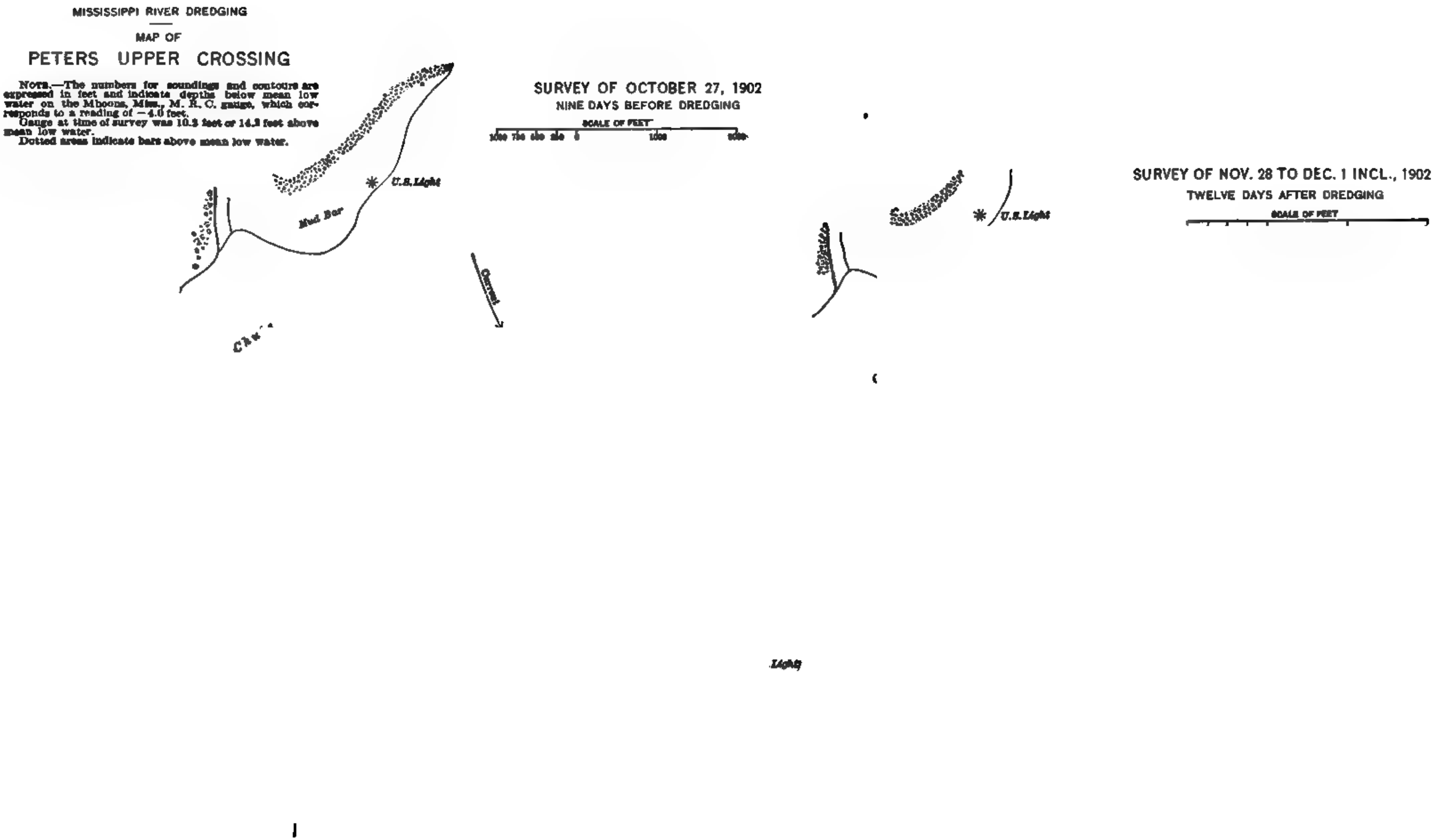



**Pl. 6.**

(Reference, p. 87.)

PLATE 7




PL. 8.
(Reference, p. 113.)

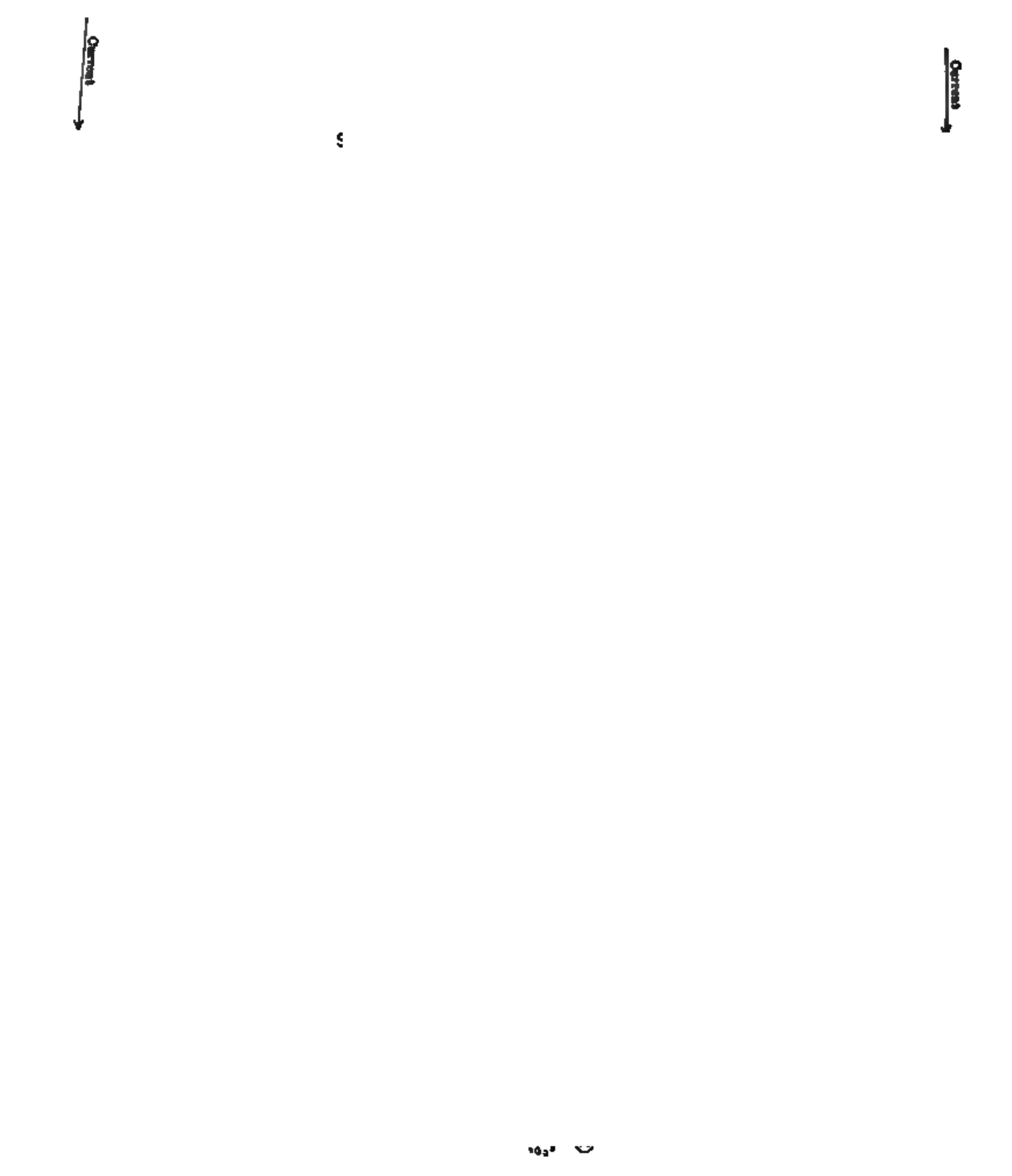
MISSISSIPPI RIVER DREDGING

MAP OF

## PETERS LOWER OR ASHLEY POINT CROSSING

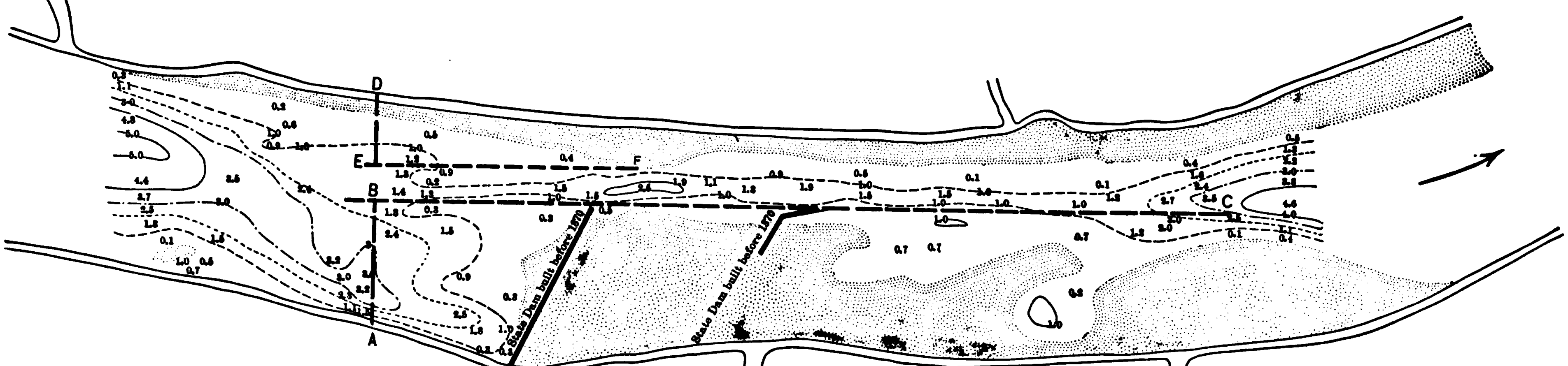
soundings and contours are
[illegible] depths below mean low
, M. R. C. gauge, which cor-
feet.
[illegible] 10.9 feet or 14.0 feet above

Dotted areas indicate bars above mean low water.




(Reference, p. 113.)

## OSAGE RIVER, MISSOURI
### BURD'S SHOAL.

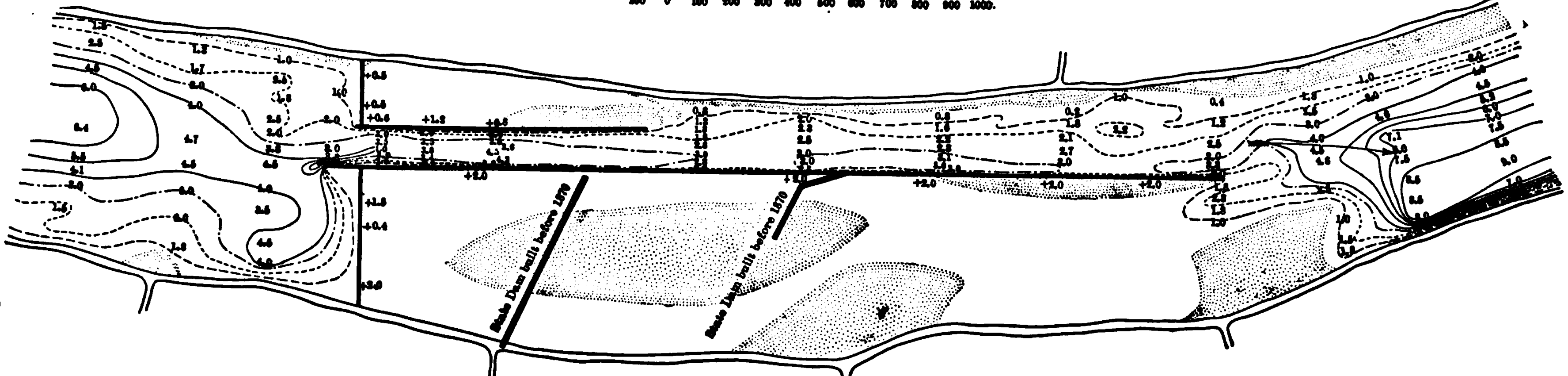



Depths are given in feet below low water.
Depths of 4 ft. and over are shown by full-line contours

CONDITION OF SHOAL IN 1872 BEFORE CONSTRUCTION OF DIKES AND LOCATION OF DIKES AS CONSTRUCTED.

Dike A B C Built in 1873, Repaired in 1874.
Dike D E F Built in 1879-80. No repairs since that time up to 1900.

SCALE OF FEET
100 0 100 200 300 400 500 600 700 800 900 1000.




CONDITION OF DIKE AND SHOAL IN 1895.

PL. 10.
(Reference, p. 141.)

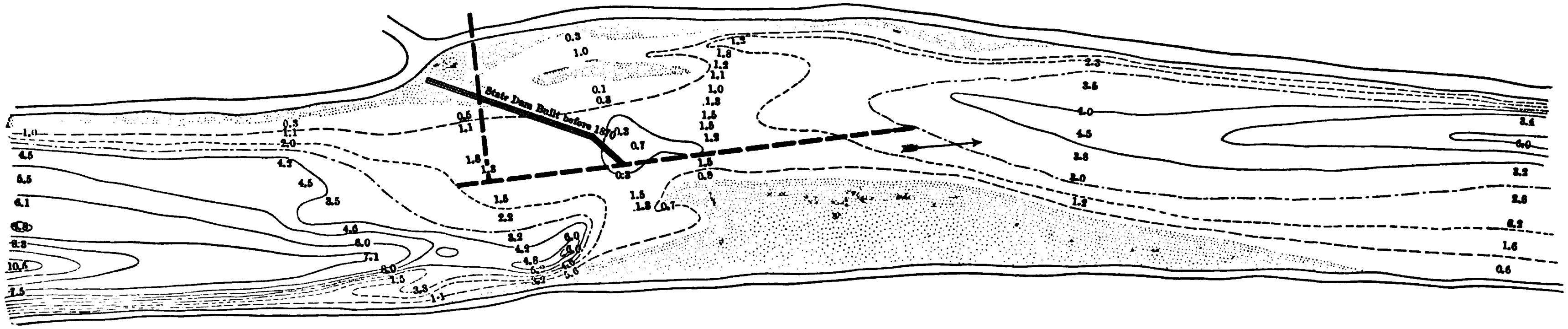

Pl. 10.

(Reference, p. 141.)

PLATE 11

# OSAGE RIVER, MISSOURI
## ROUND BOTTOM SHOAL.

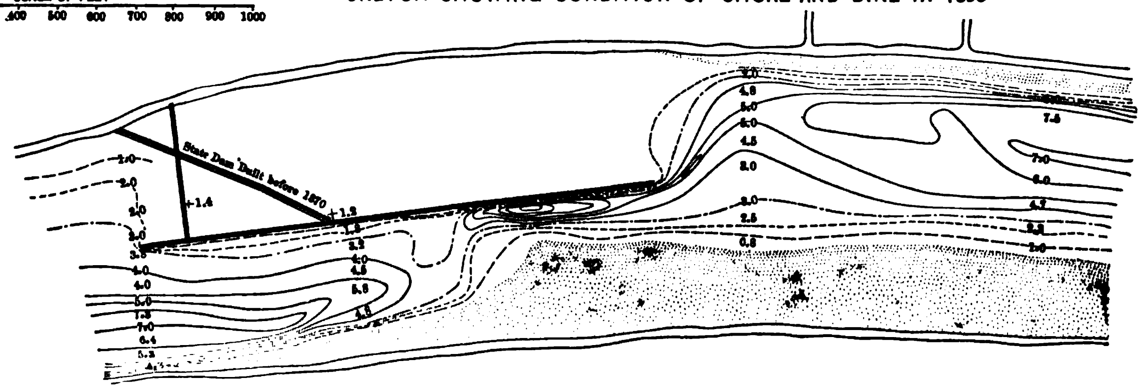
SHOAL IN 1872, SHOWING CONDITION BEFORE CONSTRUCTION OF DIKE AND LOCATION OF DIKE AS CONSTRUCTED IN 1873. NO REPAIRS OR EXTENSIONS SINCE THAT YEAR UP TO 1900




Depths are given in feet below low water.
Depths of 4 ft. and over are shown by full-line contours

SCALE OF FEET
100 0 100 200 300 400 500 600 700 800 900 1000

SKETCH SHOWING CONDITION OF SHOAL AND DIKE IN 1895

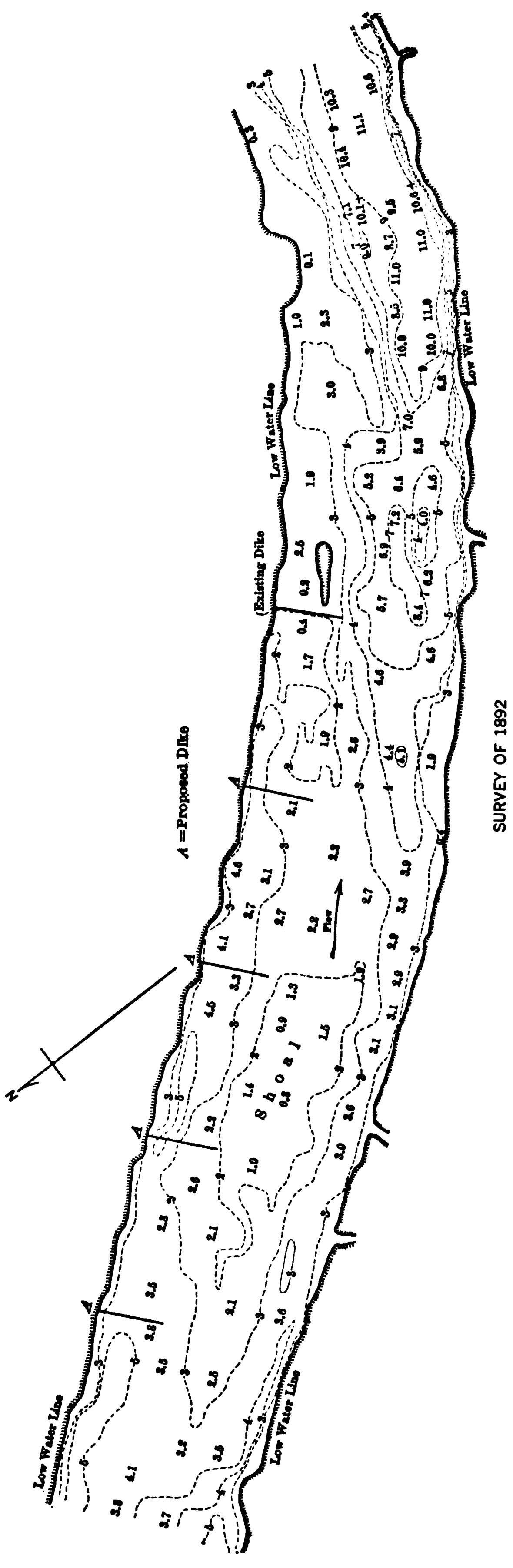


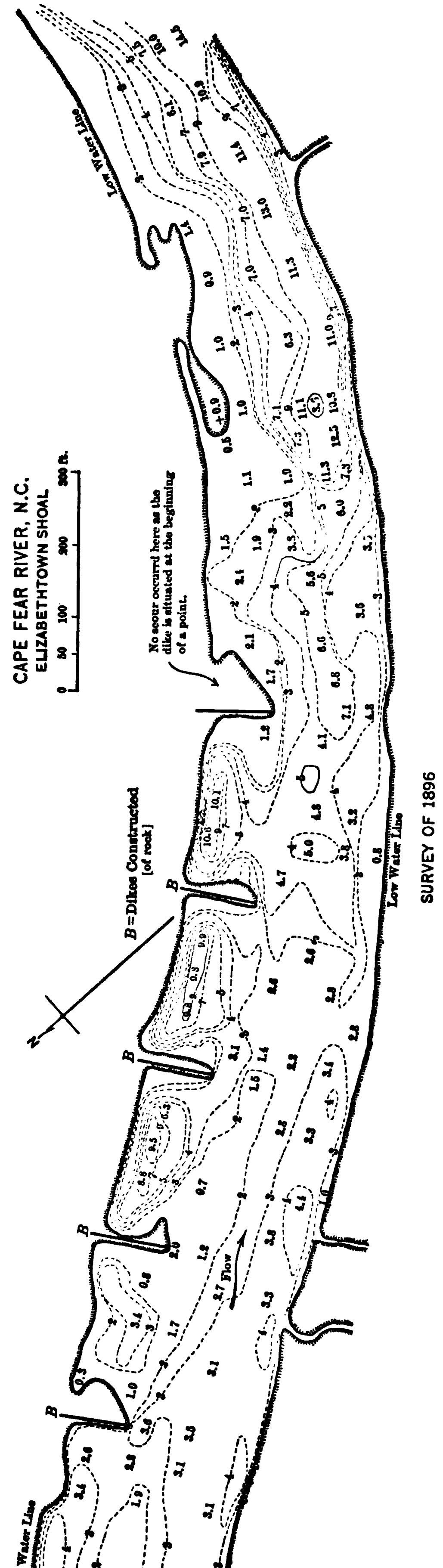
OSAGE RIVER, MISSOURI
DIXON'S SHOAL.

SHOAL IN 1872 SHOWING CONDITION BEFORE CONSTRUCTION OF DIKE AND LOCATION OF DIKE AS CONSTRUCTED

SKETCH SHOWING CONDITION OF SHOAL AND DIKE IN 1895.

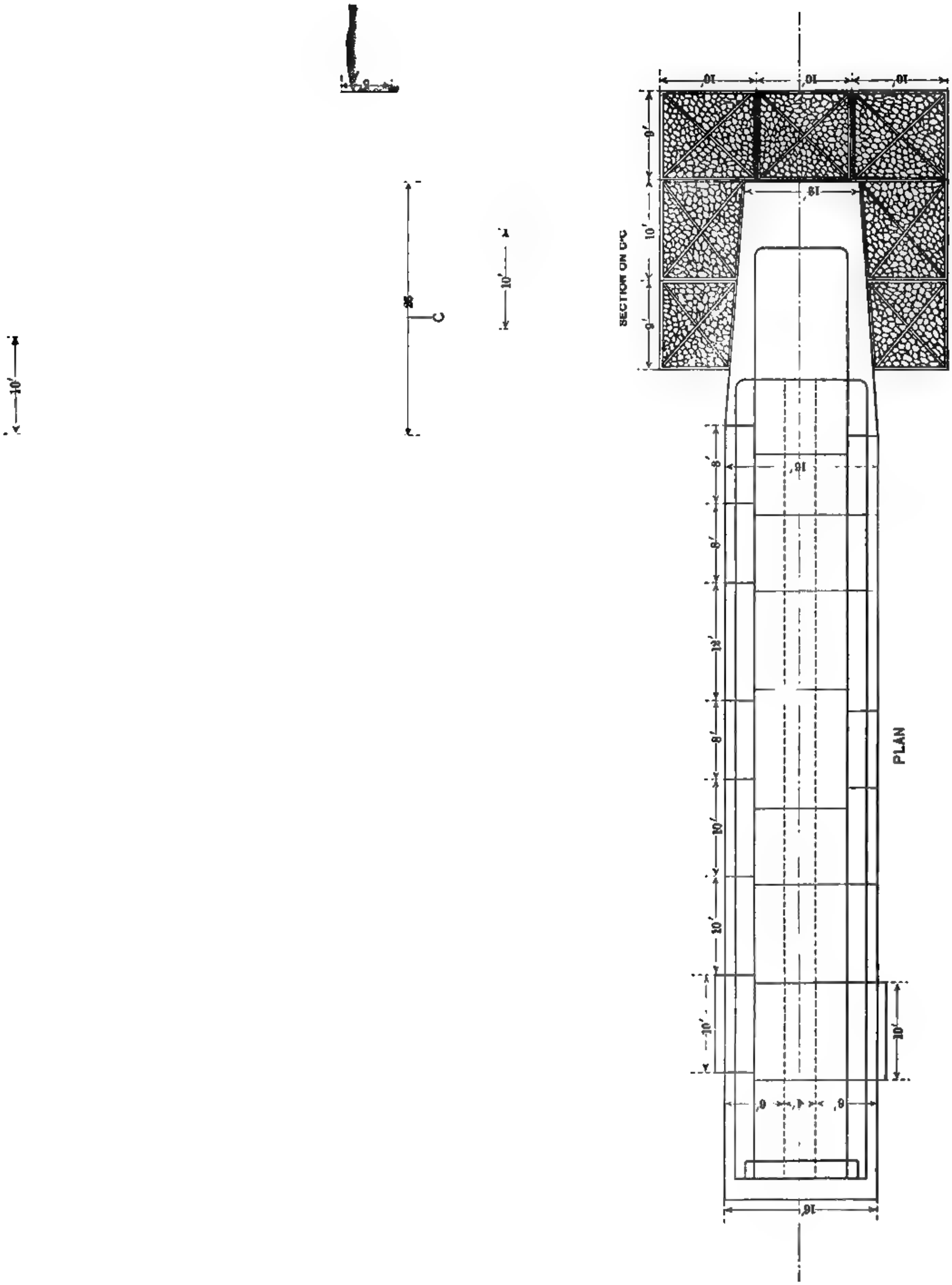
PL. 11.
(Reference, p. 141.)




Depths are given in feet below low water.

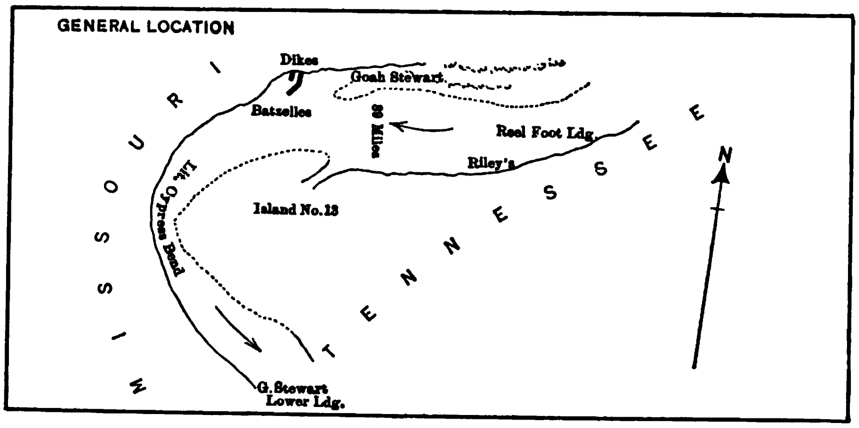
PL. 12.
(Reference, p. 144.)




TYPICAL MASONRY SPUR ON THE RIVER GANGES.
(Built 1897–1898.)

PL. 13.
(Reference, p. 155.)

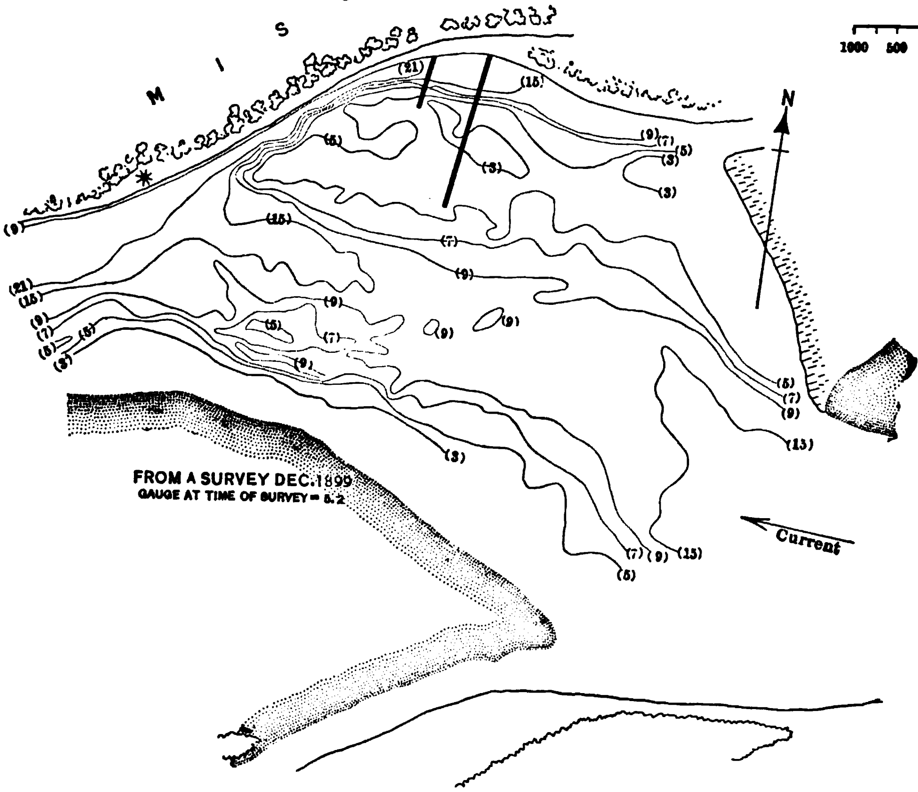
PLATE 14

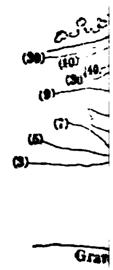


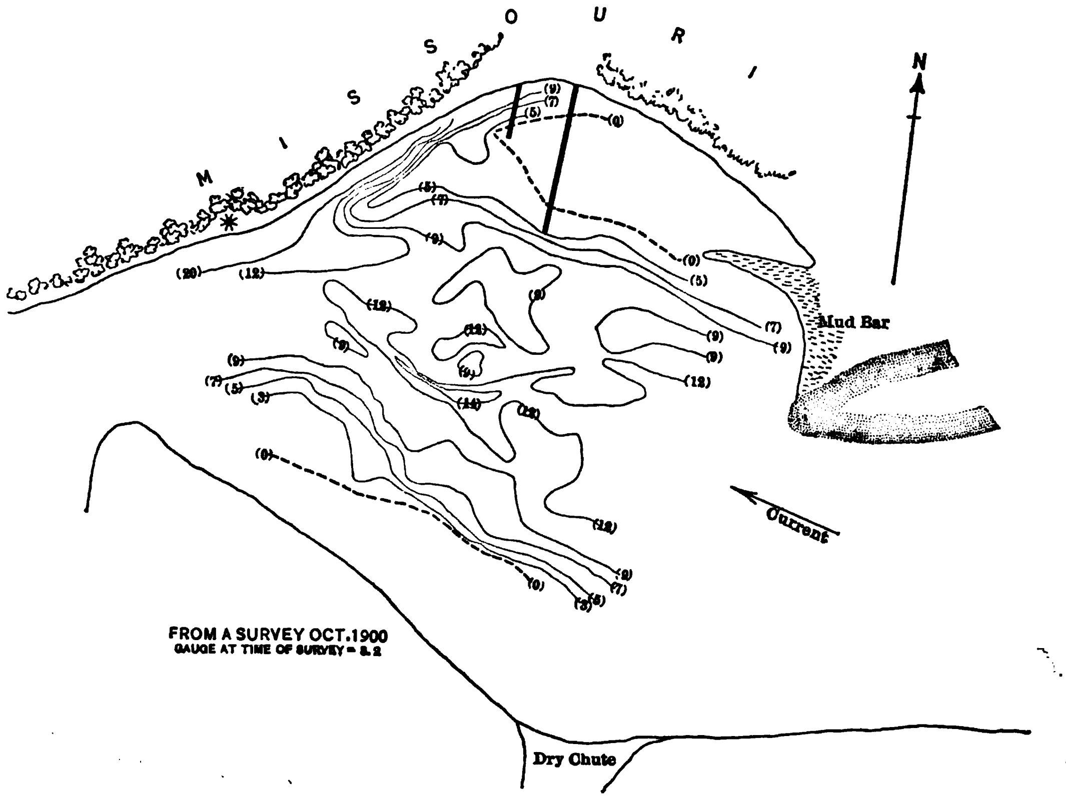

DIKES AT
CHEROKEE CRO
LOWER MISSISSIPP

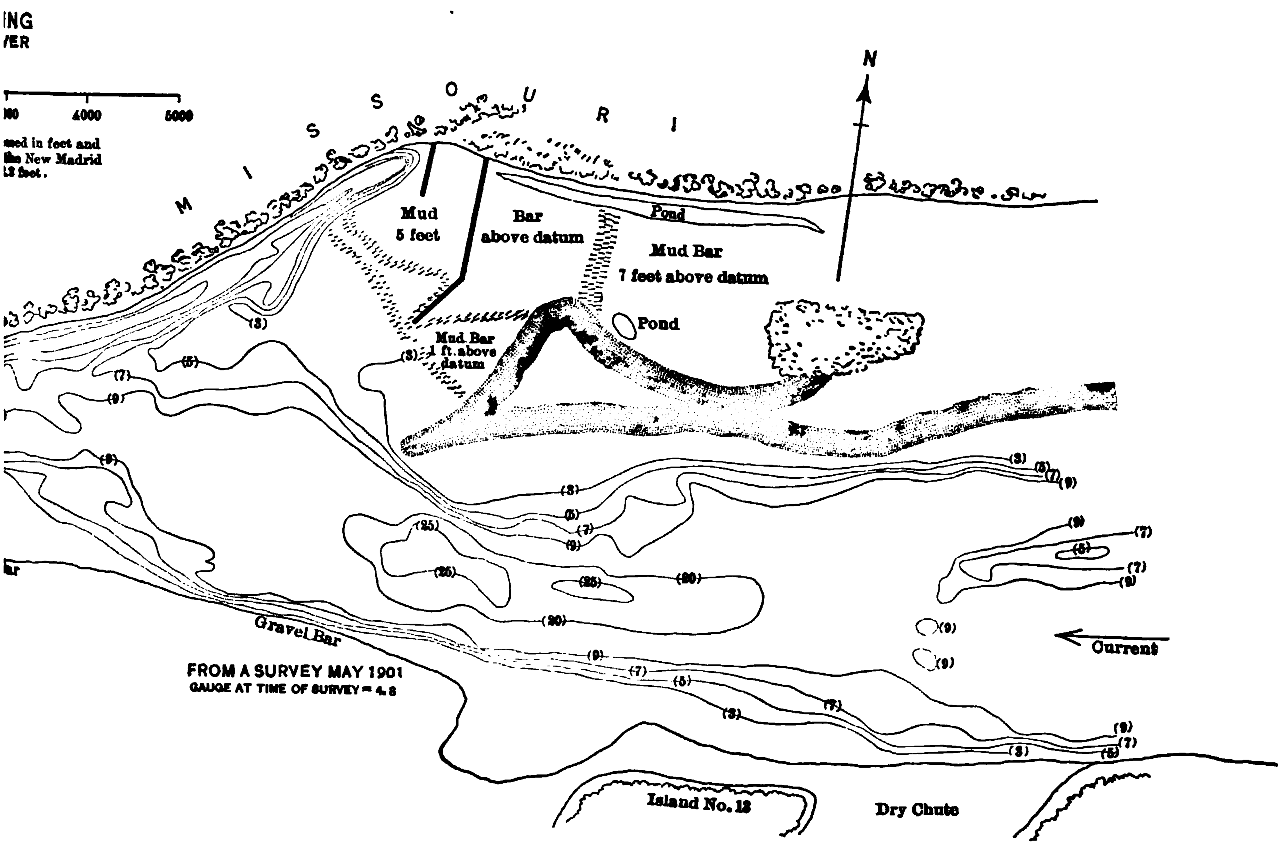
SCALE OF FEET
1000 500 0 1000 2000

Note: The numbers for contours are
indicate depths below mean low wat
gauge, which corresponds to a readi





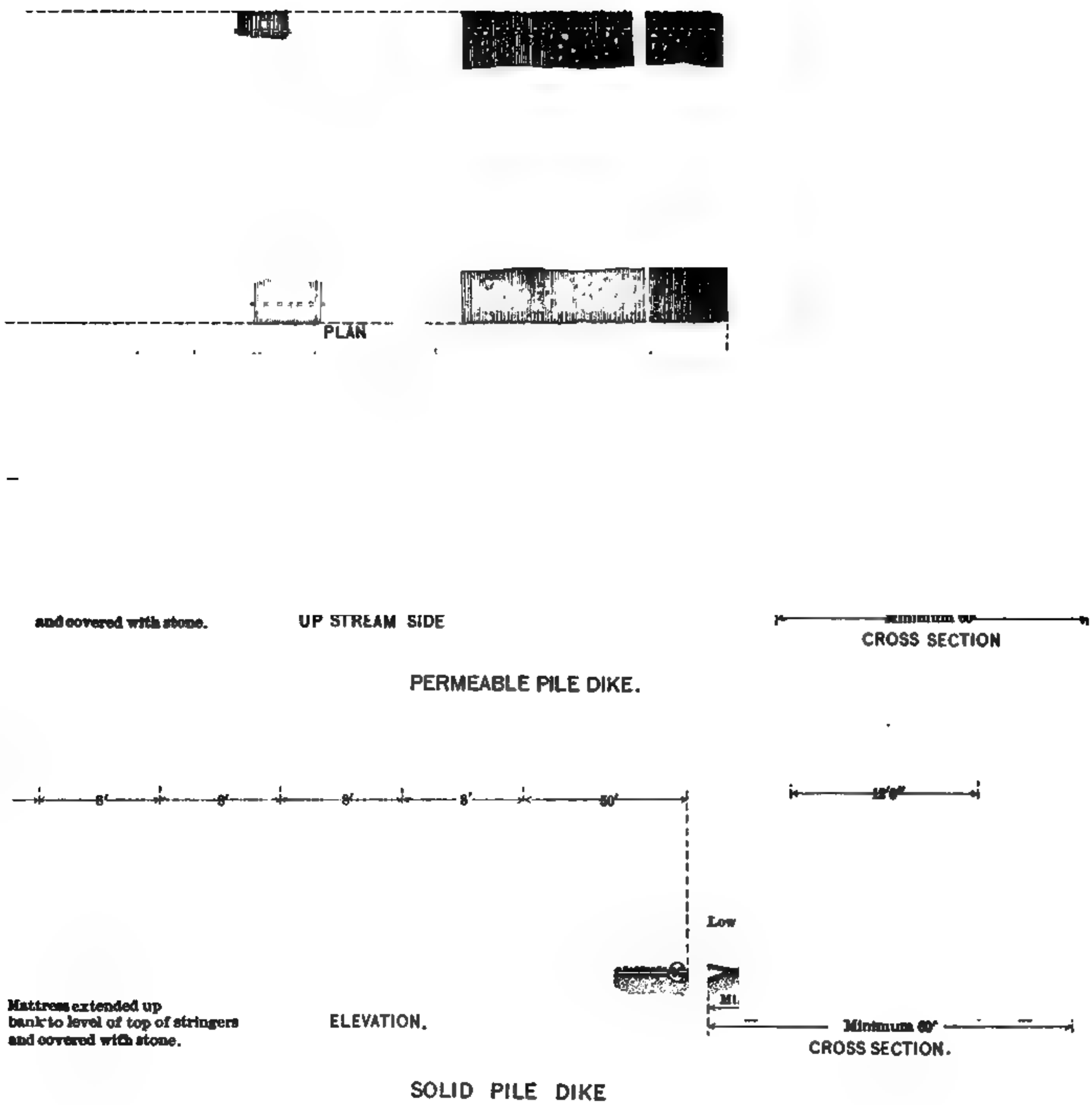




**Pl. 14.**
(Reference, p. 153.)

PLATE 15

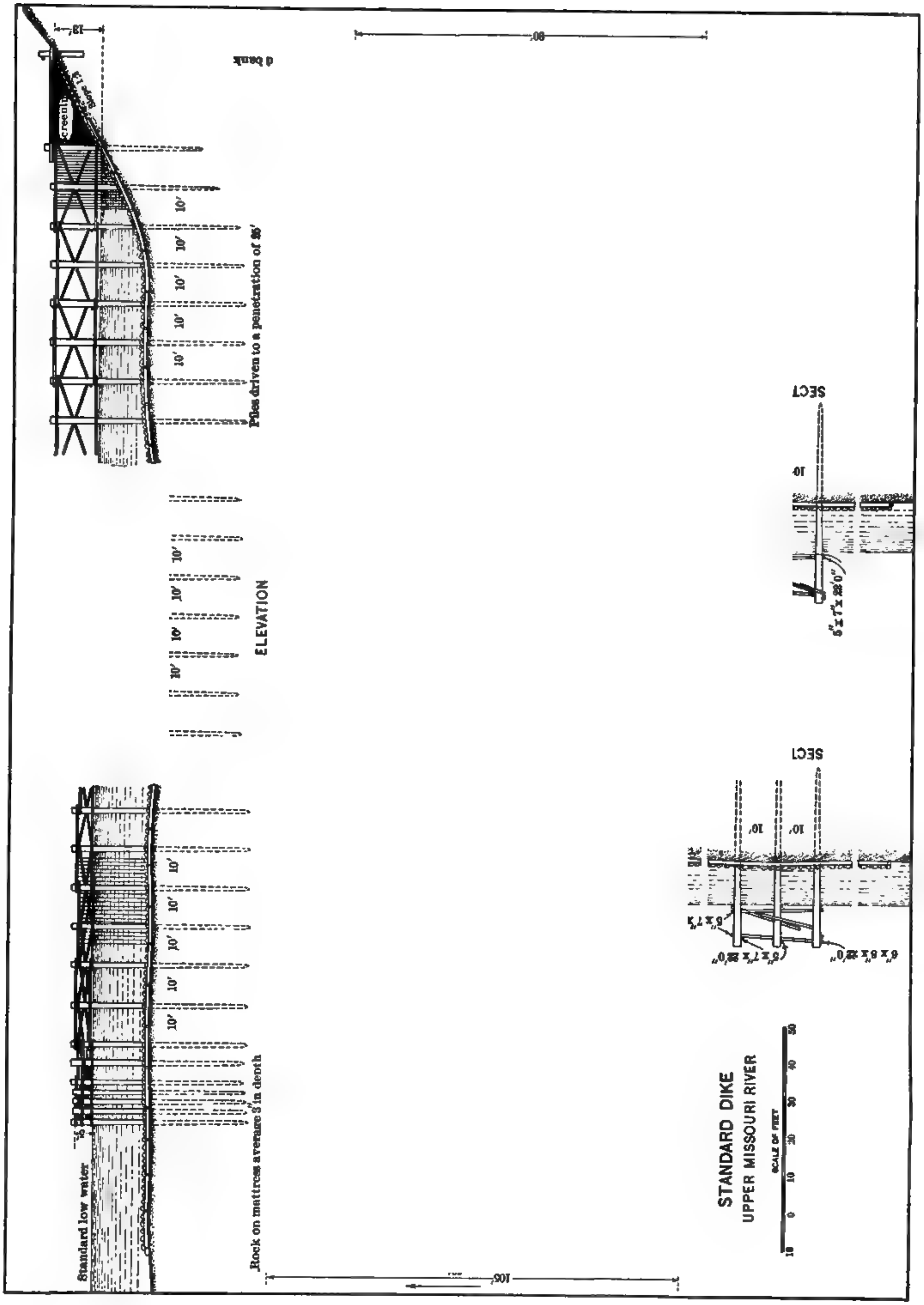

STONE DIKE.

REVETMENT.

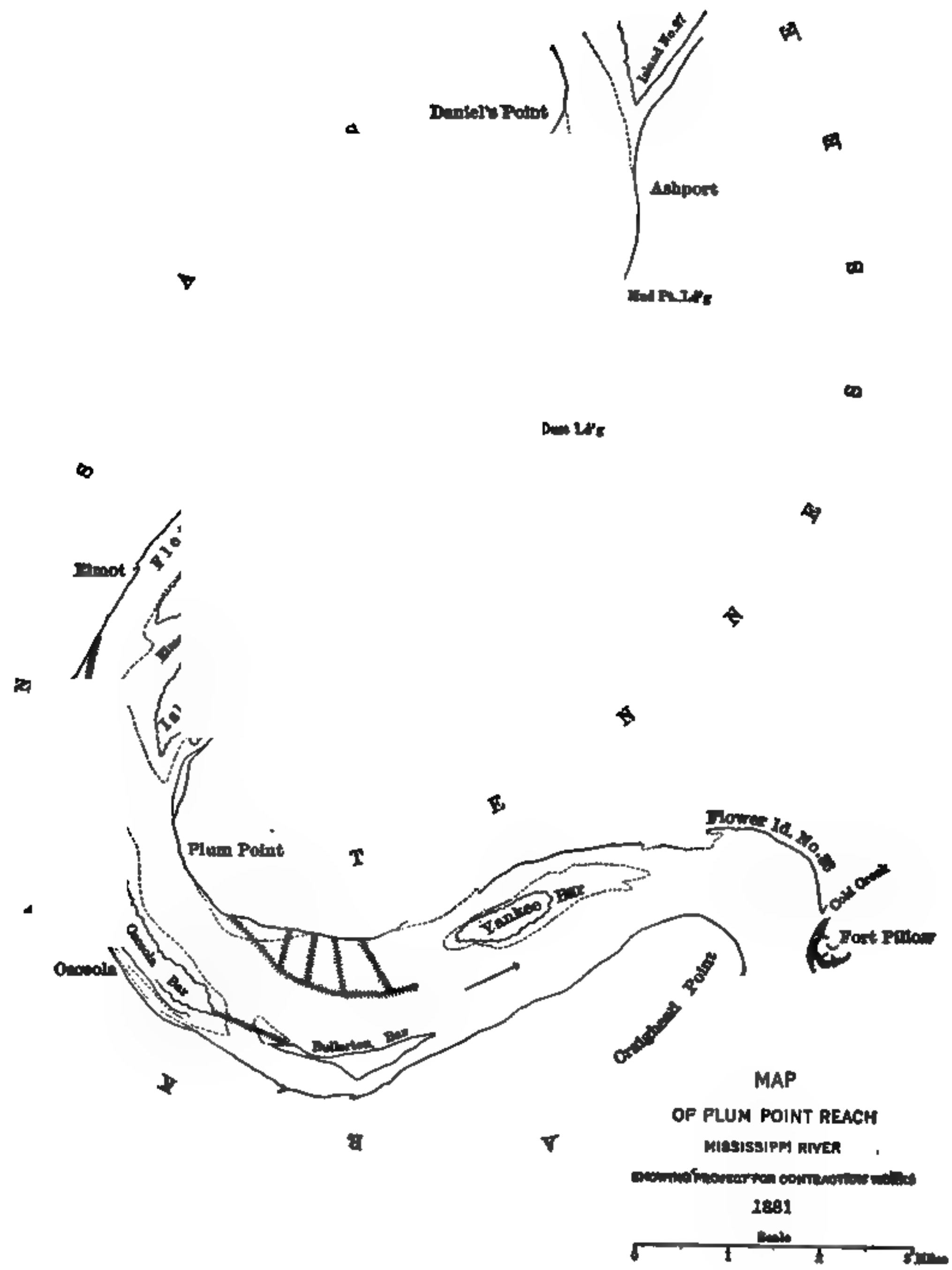



ARKANSAS RIVER, ARK.
STANDARD TYPES OF DIKES AND REVETMENT.

PL. 15.
(Reference, p. 163.)




PL. 16.
(Reference, p. 167.)




Pl. 17.
(Reference, p. 174.)

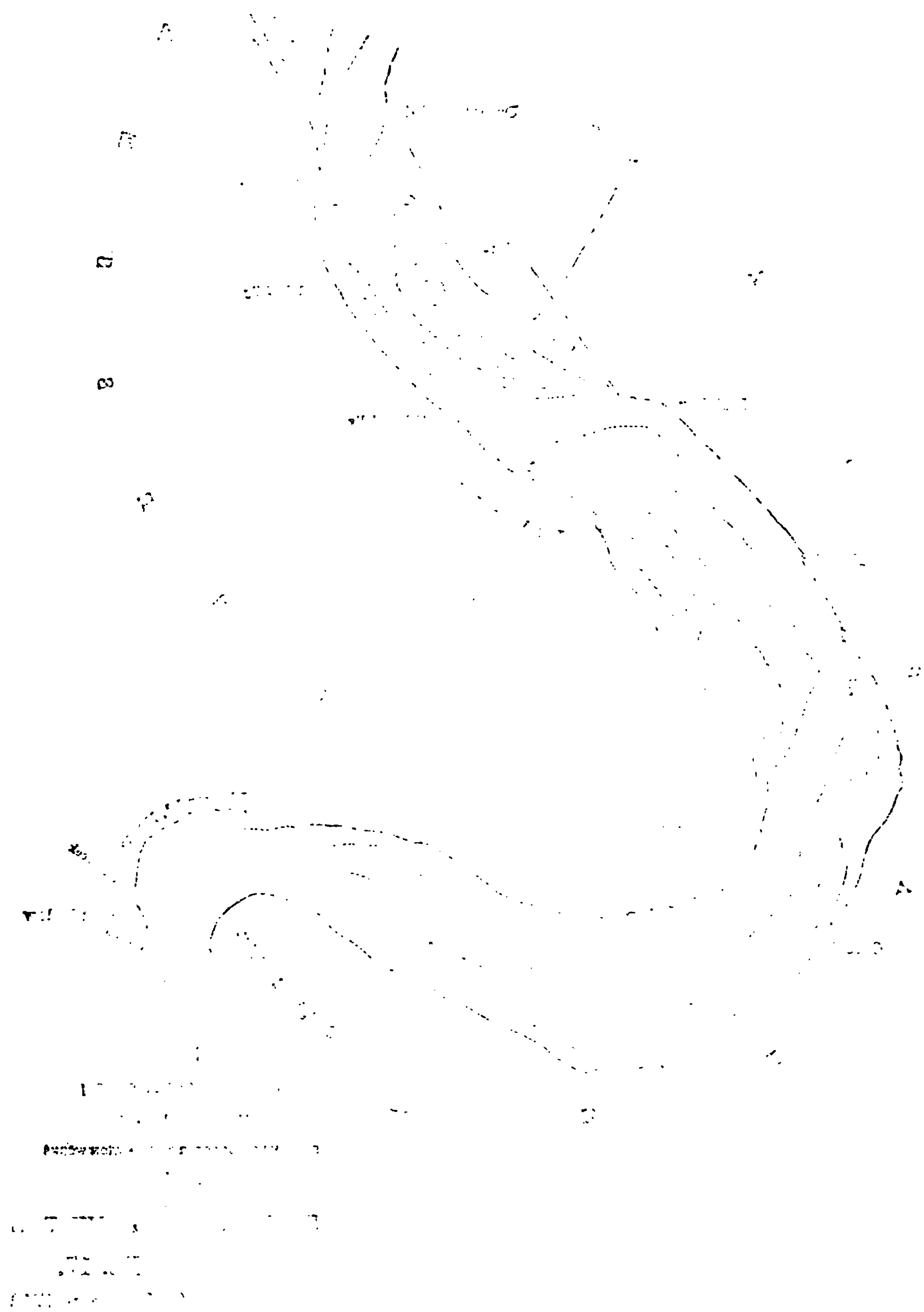
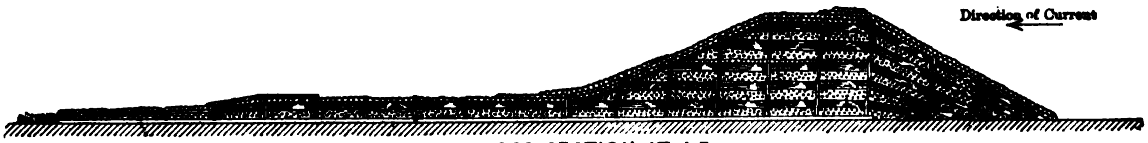
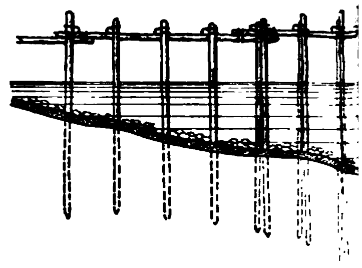
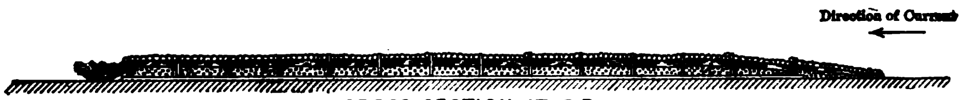


PLATE 18

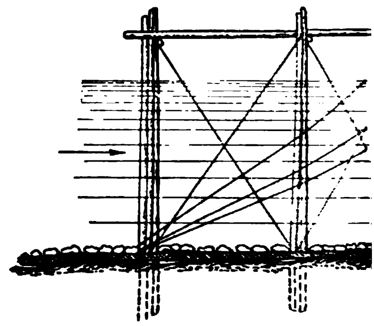


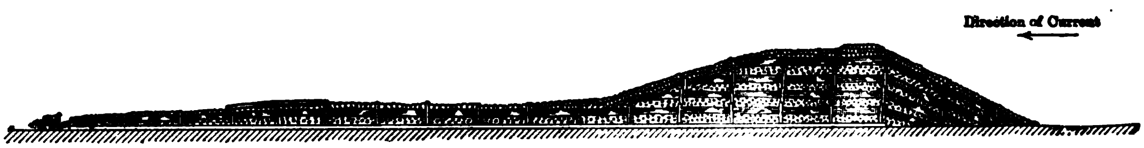

CROSS SECTION AT A-B



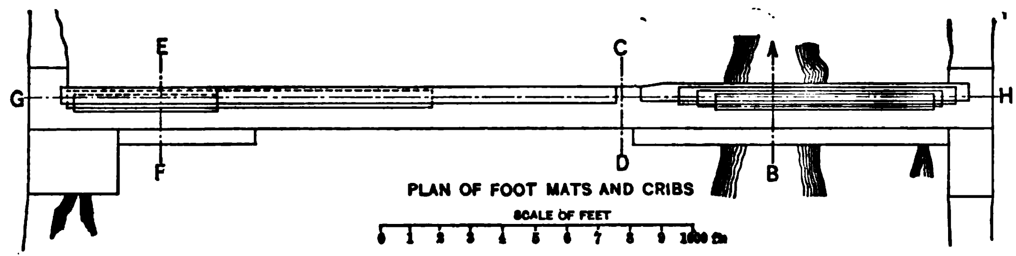




CROSS SECTION AT C-D

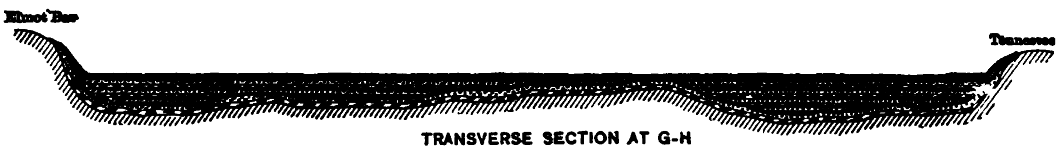





CROSS SECTION AT E-F




PLAN OF FOOT MATS AND CRIBS




TRANSVERSE SECTION AT G-H

HORIZONTAL SCALE

0 1 2 3 4 5 6 7 8 9 1000 ft.

VERTICAL SCALE

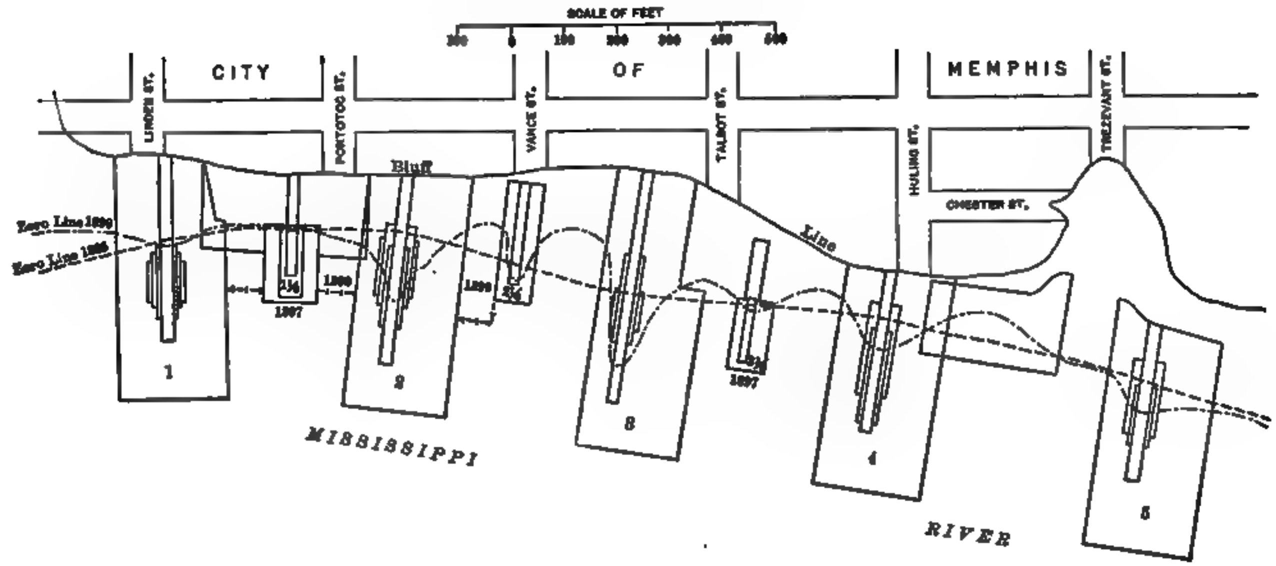
0 10 20 30 40 ft.

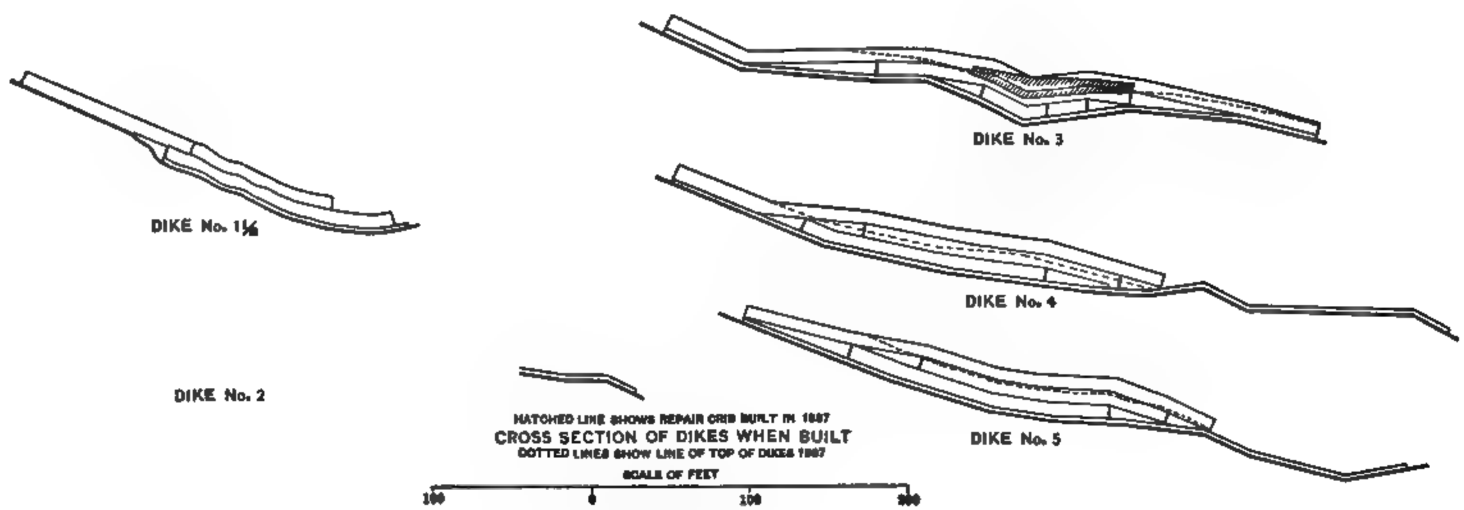
PLUM POINT REACH

Details of

GOLD DUST DAM.

PLAN SHOWING POSITION OF DIKES

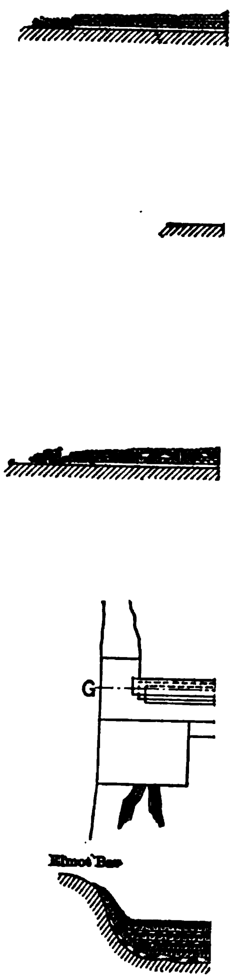


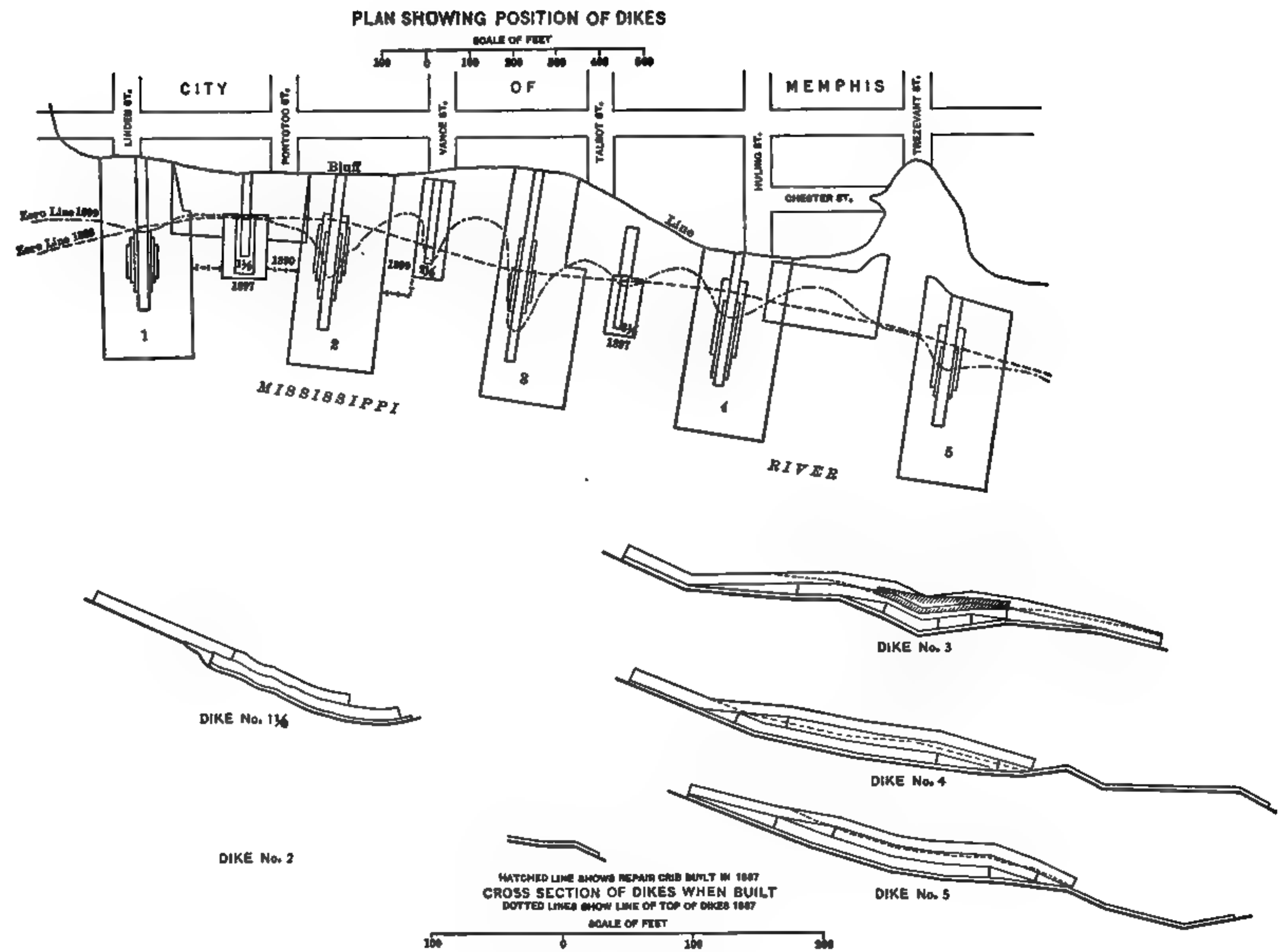




SPUR DIKES AT MEMPHIS, TENNESSEE

Pl. 19.

(Reference, p. 192.)


G



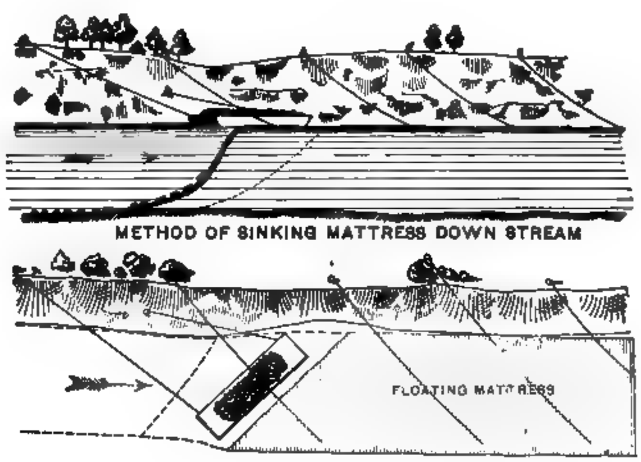

SPUR DIKES AT MEMPHIS, TENNESSEE

PL. 19.

(Reference, p. 192.)

PLATE 20

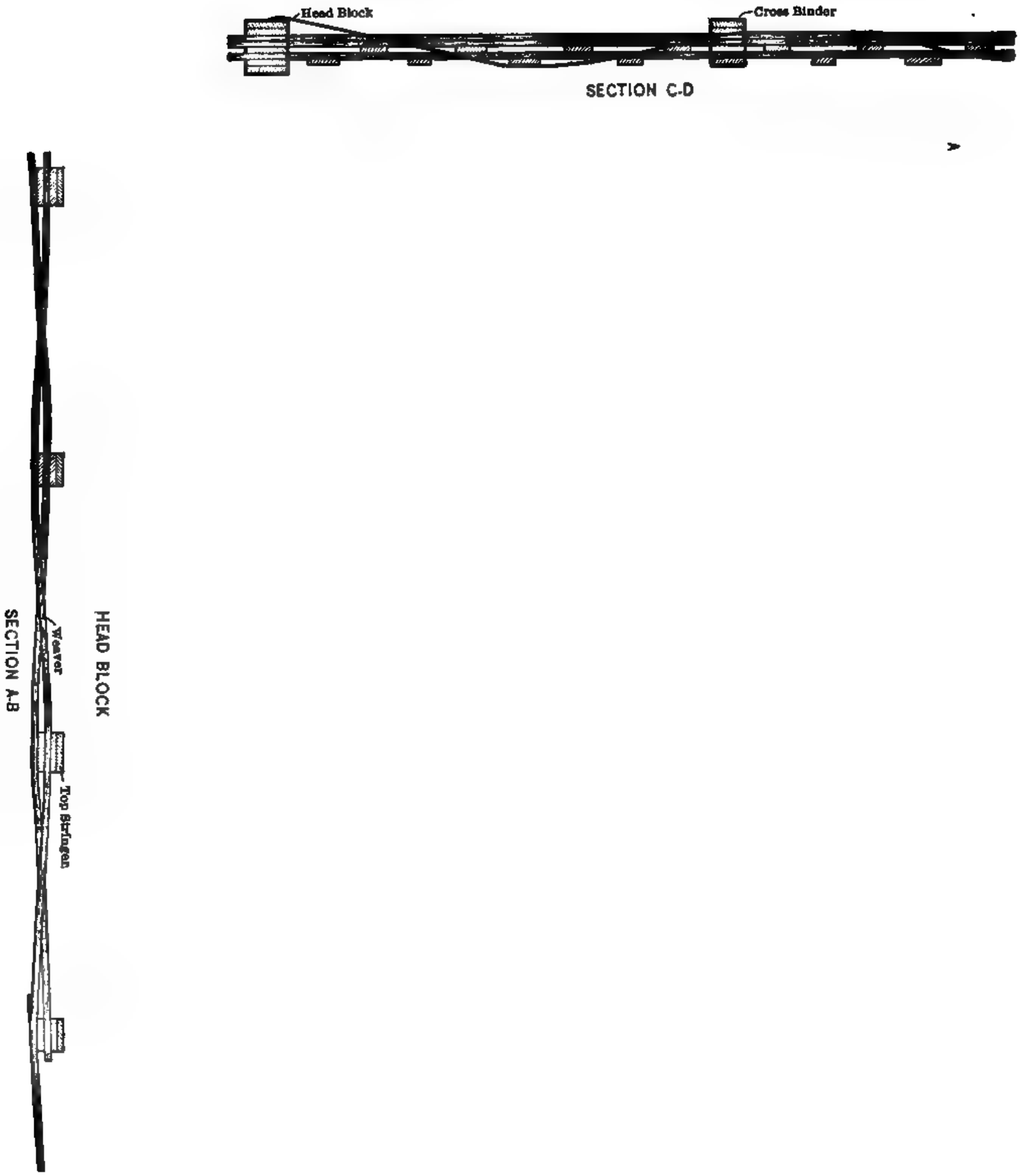

PLAN AND SECTION




Pole and Wire Mattresses.

Methods of Sinking Mattresses.

GENERAL DETAILS OF WOVEN MATTRESSES




GENERAL DETAILS OF LUMBER MATTRESS.

PL. 20.

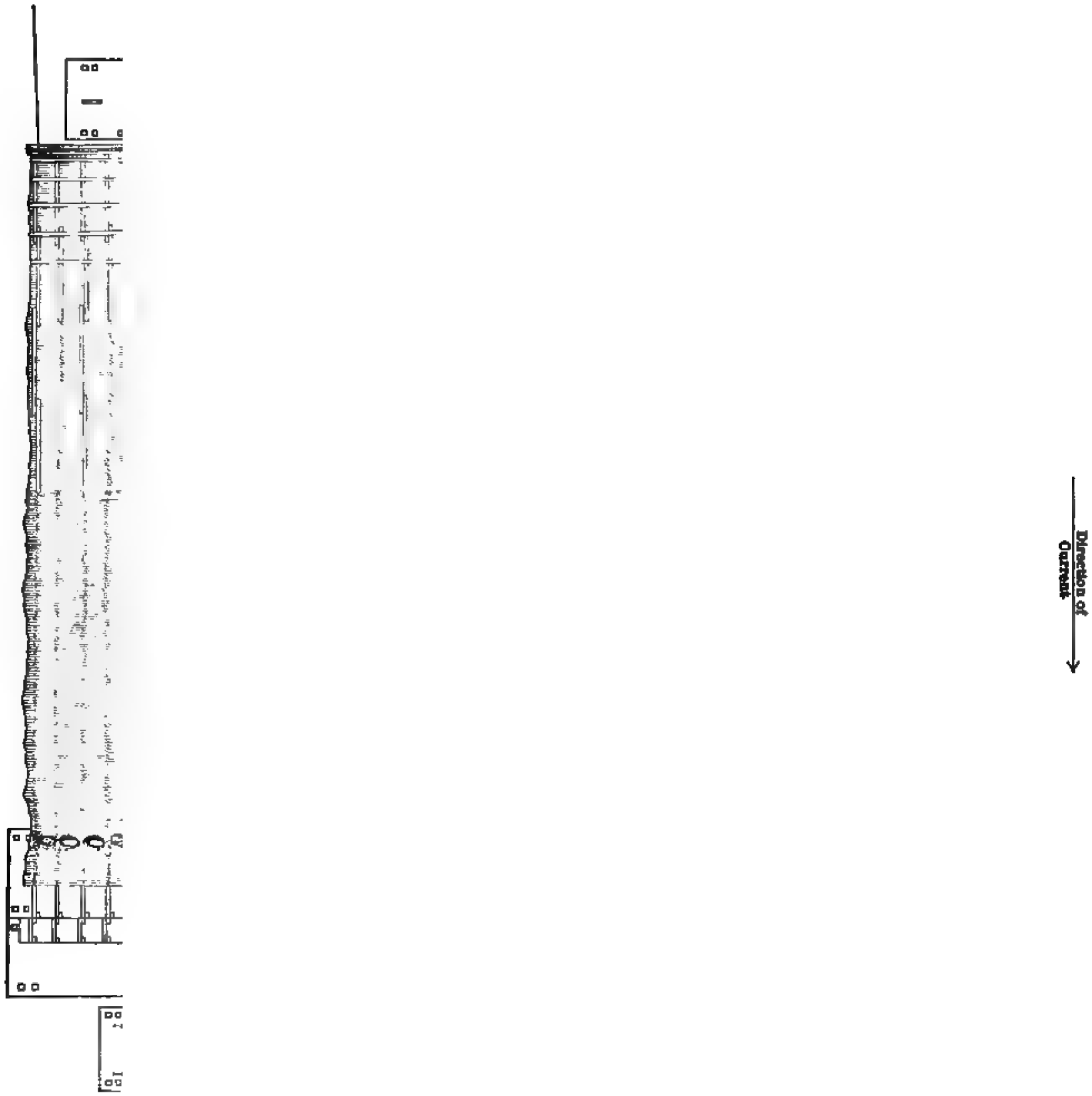
(Reference, pp. 188, 193, 215.)


SKETCH SHOWING ARRANGEMENT OF CHAINS FOR STOPPING DRIFT.

Fig. a

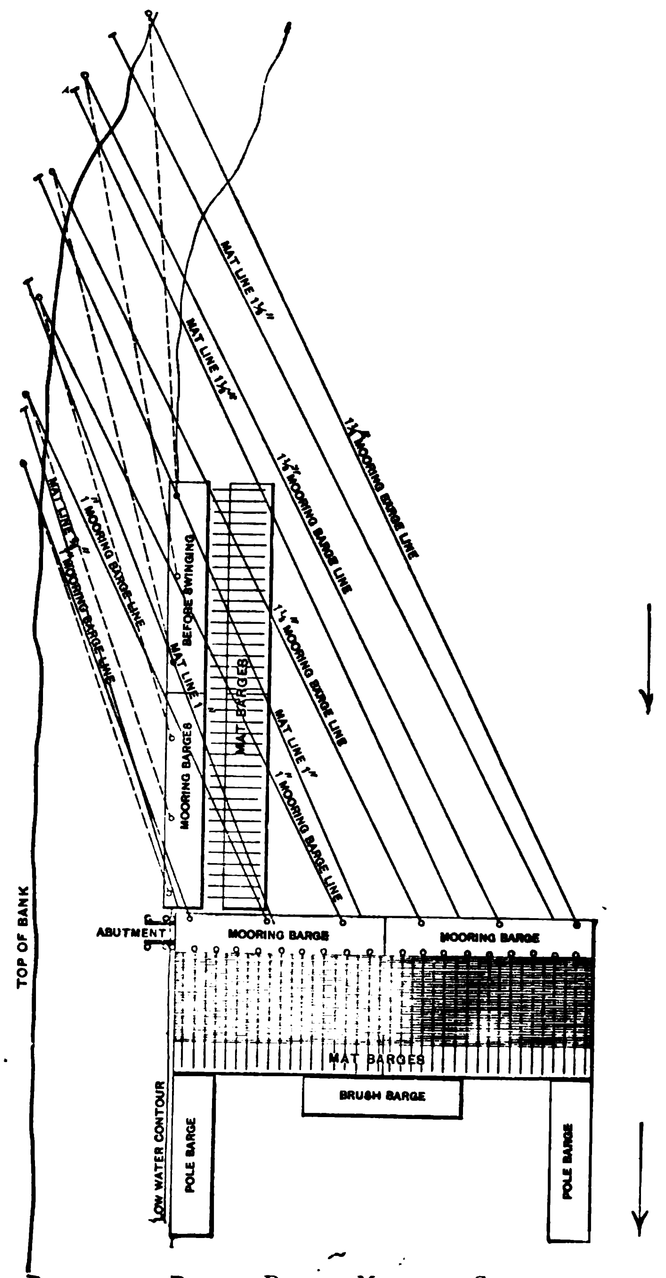
**PL. 21.**

(Reference, pp. 202, 204.)



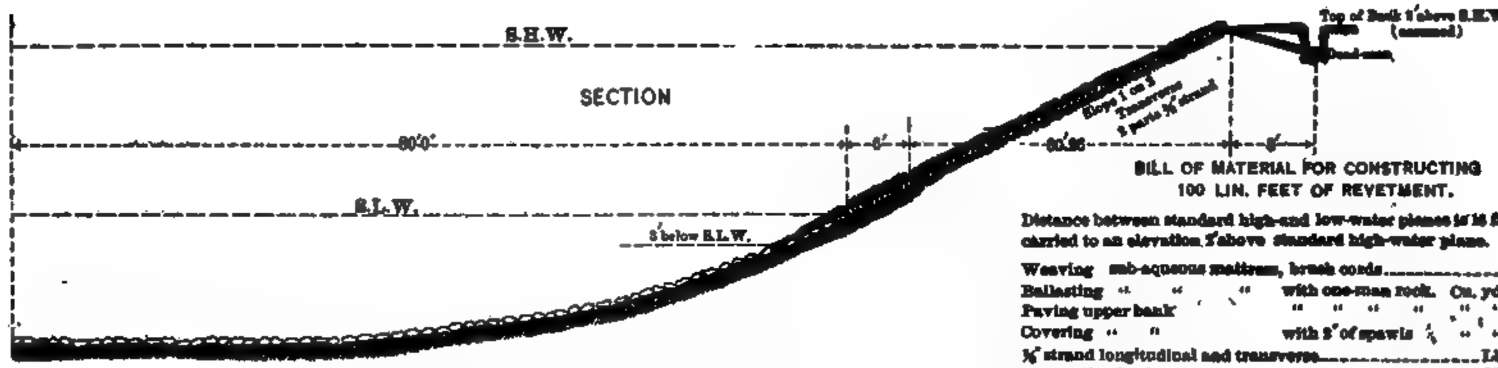

SKETCH
SHOWING CONSTRUCTION OF
FASCINE MATTRESS
SCALE OF FEET
0 5 10 20 30 40 50 60 70 80 90 100
(See also Pl. 23.)

PL. 22.
(Reference, p. 204.)




POSITION OF BARGES DURING MATTRESS CONSTRUCTION.
(See also Pl. 22.)

PL. 23.
(Reference, pp. 203, 204, 215.)




NOTE.

Between limits of 3 ft. above and 8 ft. below S.L.W. spawls sufficient to fill interstices in mat and fair up selvage edge on the slope are distributed prior to paving.

**BILL OF MATERIAL FOR CONSTRUCTING 100 LIN. FEET OF REVETMENT.**

Distance between standard high-and low-water planes is 16 ft., and paving carried to an elevation 2' above standard high-water plane.

| | | |
|---|---|---|
| Weaving sub-aqueous mattress, brush cords | | 60 |
| Ballasting " " " with one-man rock. | Cu. yds. | 75 |
| Paving upper bank " " " " | " | 150 |
| Covering " " with 2" of spawls | " | 34 |
| ⅜" strand longitudinal and transverse | Lbs. | 700 |
| 4 part clips for fastenings | No. | 42 |
| 2 " " " " | " | 12 |
| Dead-man pile heads 4 x 12 diam. long-leaf yellow pine or white oak, or, stone blocks, 25" x 12" x 12" | No. | 6 |

NOTE.

Quantity of stone on upper bank varies with height between standard high- and low-water planes, the height of bank, the stage at which work is done, and with the grade of the slope, the paving being 12" thick at standard low-water and 12", 8" and 16" thick, respectively, at the top of 1 on 2, 1 on 2½ and 1 on 3 slopes, with 2" of spawls on top.

:ROSS-SECTION

OF

REVETMENT,

'RI RIVER

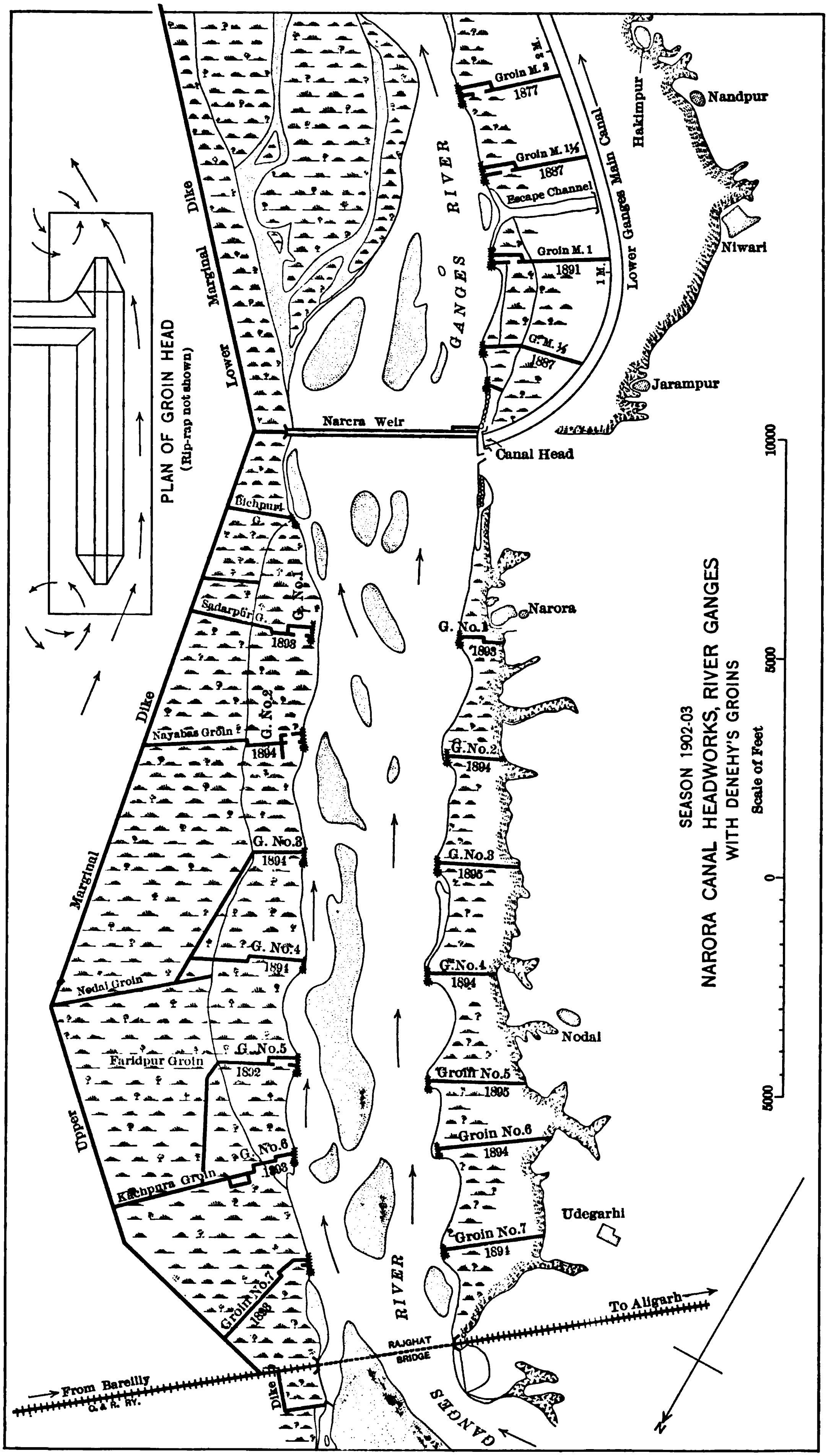
Pl. 24.

(Reference, pp. 218, 220, 222.)

Pl. 25.
(Reference, pp. 225, 227.)

PLATE 26

Pl. 27.
(Reference, p. 228.)




SEASON 1902-03
NARORA CANAL HEADWORKS, RIVER GANGES
WITH DENEHY'S GROINS

Pl. 27.
(Reference, p. 228.)

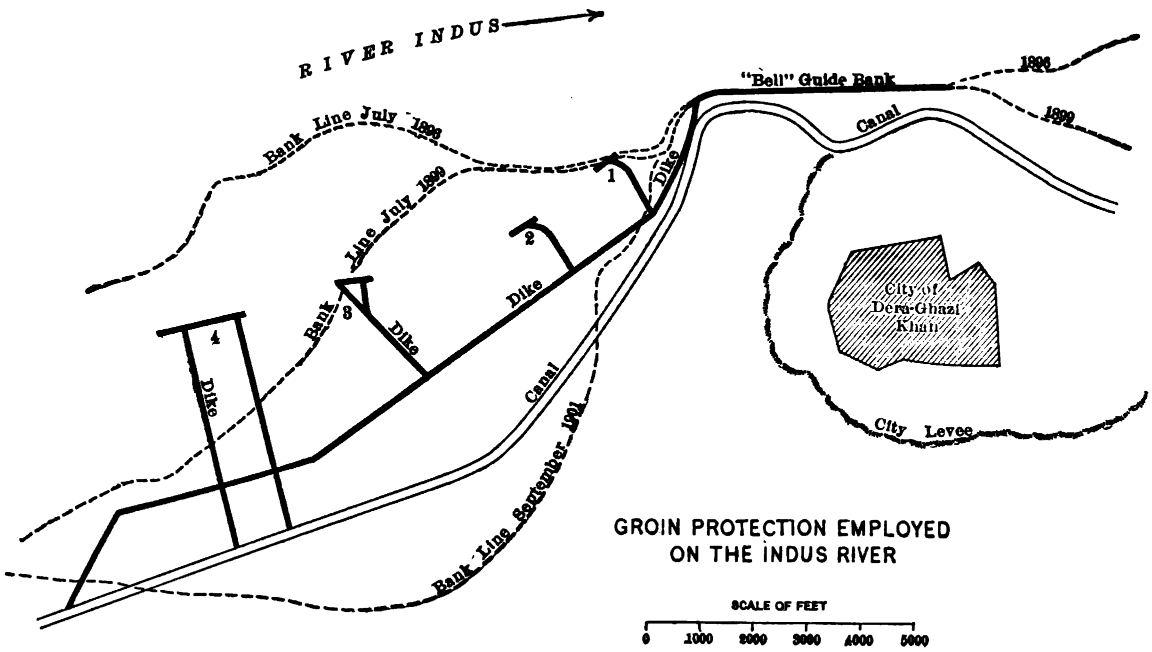



**Fig. 1.**

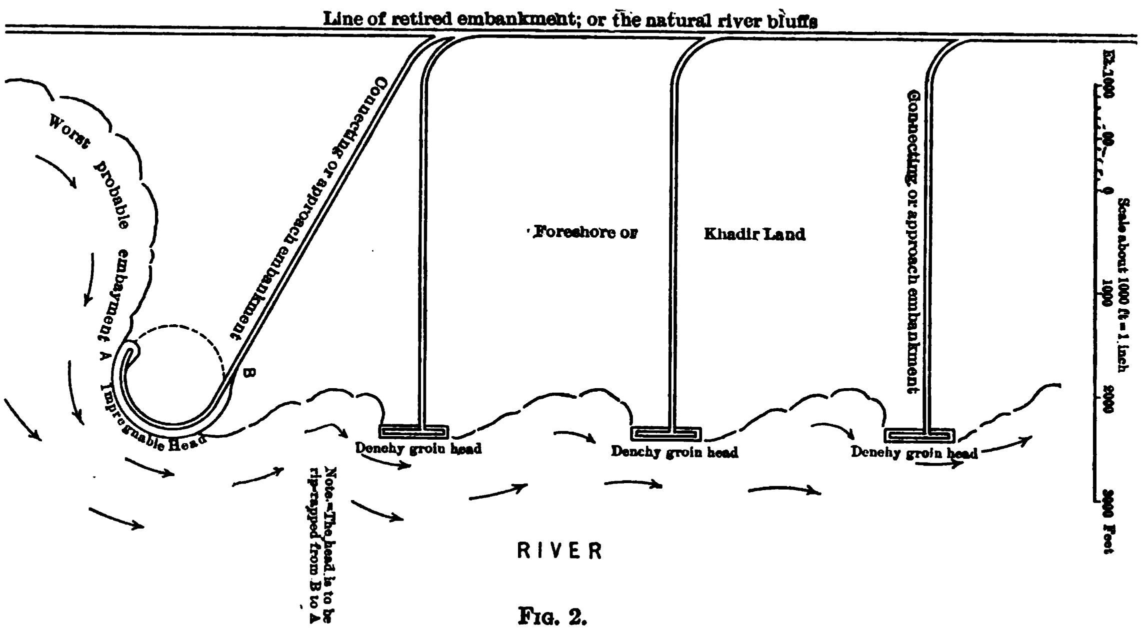


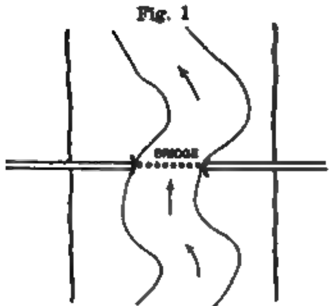

**Fig. 2.**

Pl. 28.

(Reference, p. 228.)

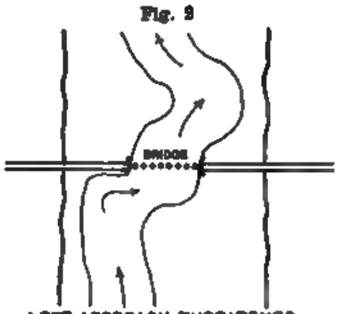
## SKETCHES ILLUSTRATING
### ATTACK ON AN UNPROTECTED BRIDGE

Fig. 1



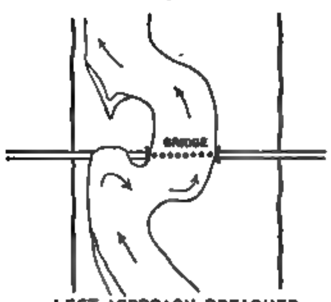

BRIDGE ERECTED.

Fig. 2



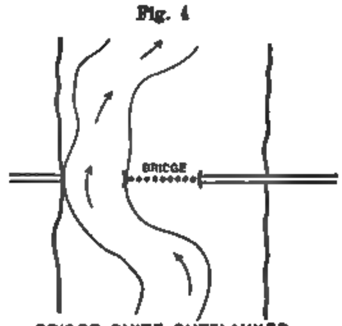

LEFT APPROACH THREATENED.

Fig. 3



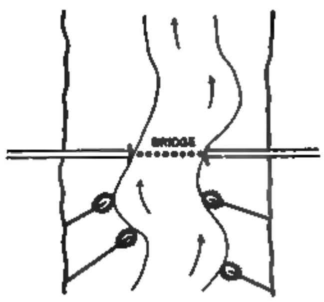

LEFT APPROACH BREACHED.

Fig. 4



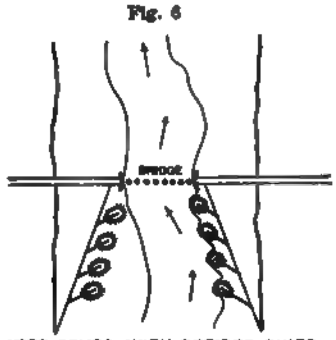

BRIDGE QUITE OUTFLANKED.

Fig. 5



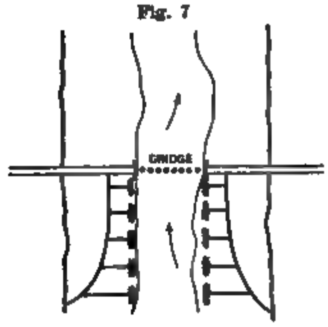

EARLY SPUR TRAINING.

Fig. 6



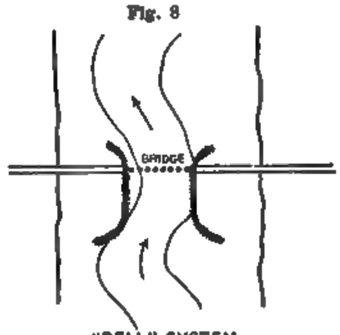

LATER SPURS, WITH RETIRED DIKES.

Fig. 7




.NEHEY'S SPURS, WITH RETIRED DIKES.

Fig. 8




"BELL" SYSTEM.

## EFFECT OF ATTACK
### ON A BRIDGE PROTECTED BY BELL GUIDE BANKS

Fig. 9

River attacks
Left guide bank.

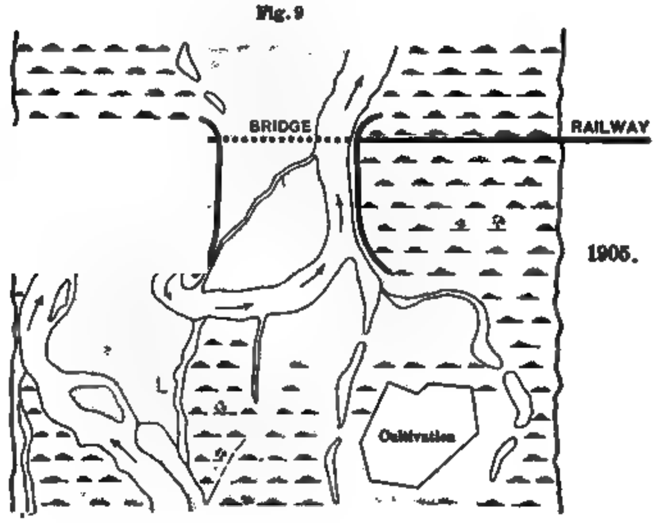



Fig. 10

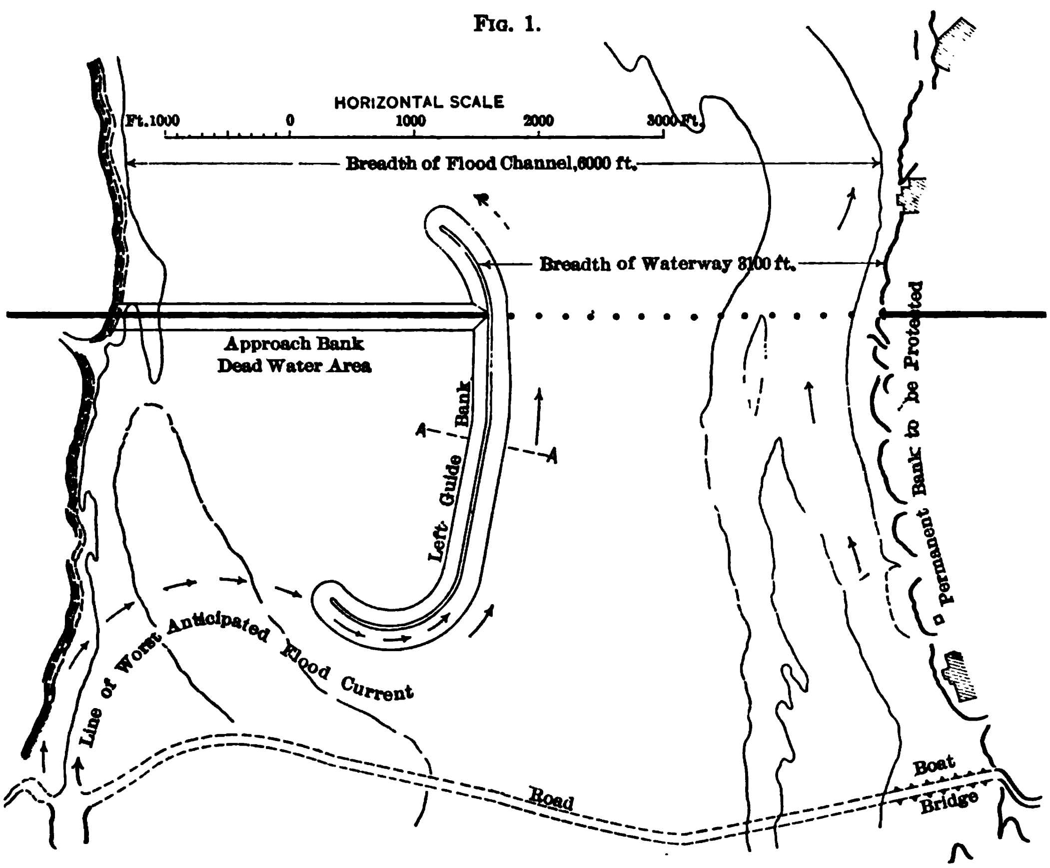
Fig. 11

.TYPES OF RIVER TRAINING IN INDIA.

PL. 29.
(Reference, p. 229.)

PLATE 30

Fig. 1.




GENERAL LOCATION PLAN

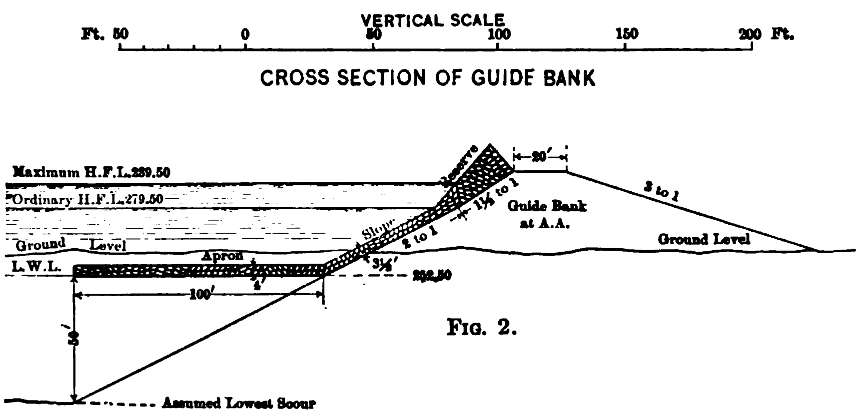



Fig. 2.

BRIDGE OVER THE GANGES RIVER AT ALLAHABAD, AS ORIGINALLY PROPOSED

Pl. 31.

(Reference, pp. 231, 235.)

PLATE 32

# THE LENGTH OF BRIDGE FOR A GIVEN DISCHARGE

Fig.1

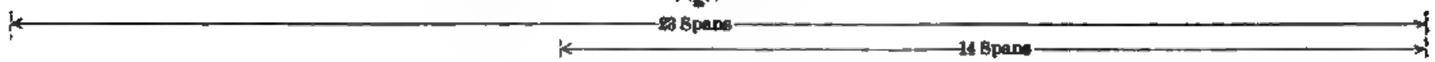



Fig.3

Fig.2

When $L=2l$, $A=4a$

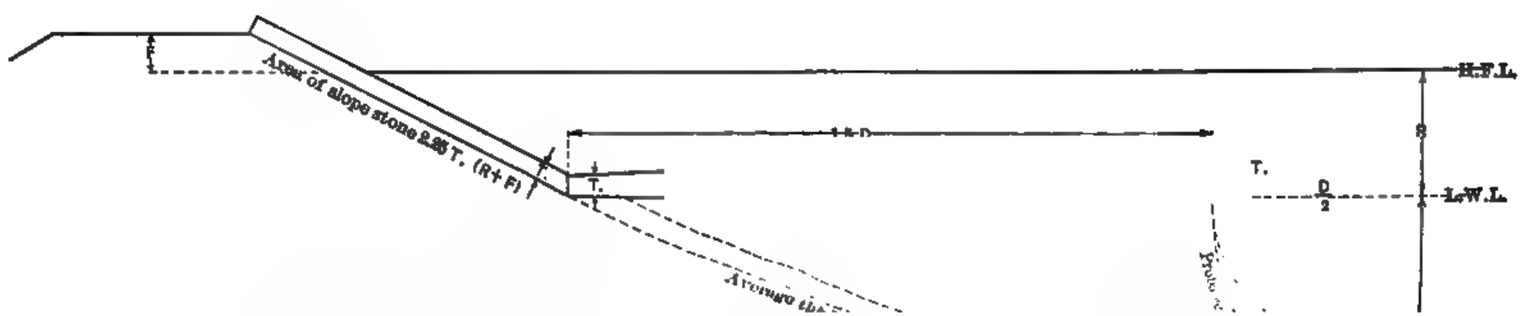
Diagram showing the relative advantages of wide and narrow spans when the piers are protected with rip-rap.

Probable Bed, S, Fig.1

Form when piers are rip-rapped

Assumed form for calculating depth of foundations

All three sections to give equal discharges. But in ______ scour takes place in only one third of the number of bridge spans.



F.——Freeboard.
R.——Rise of Flood.
D.——Deepest known scour.
T.——Thickness of slope stone.
Area of slope stone.——2.25 T. (R+ F)
Area of apron stone.——2.82 D.T.
Width of apron.——1.50 D.
Mean thickness of apron.——1.88 T.
Inside thickness of apron.——T.
Outside thickness of apron.——2.76 T.
Inclination of slope stone.——2 to 1.
Desired inclination of apron stone.——2 to 1.

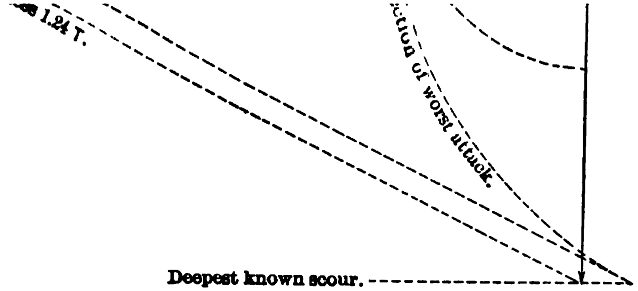
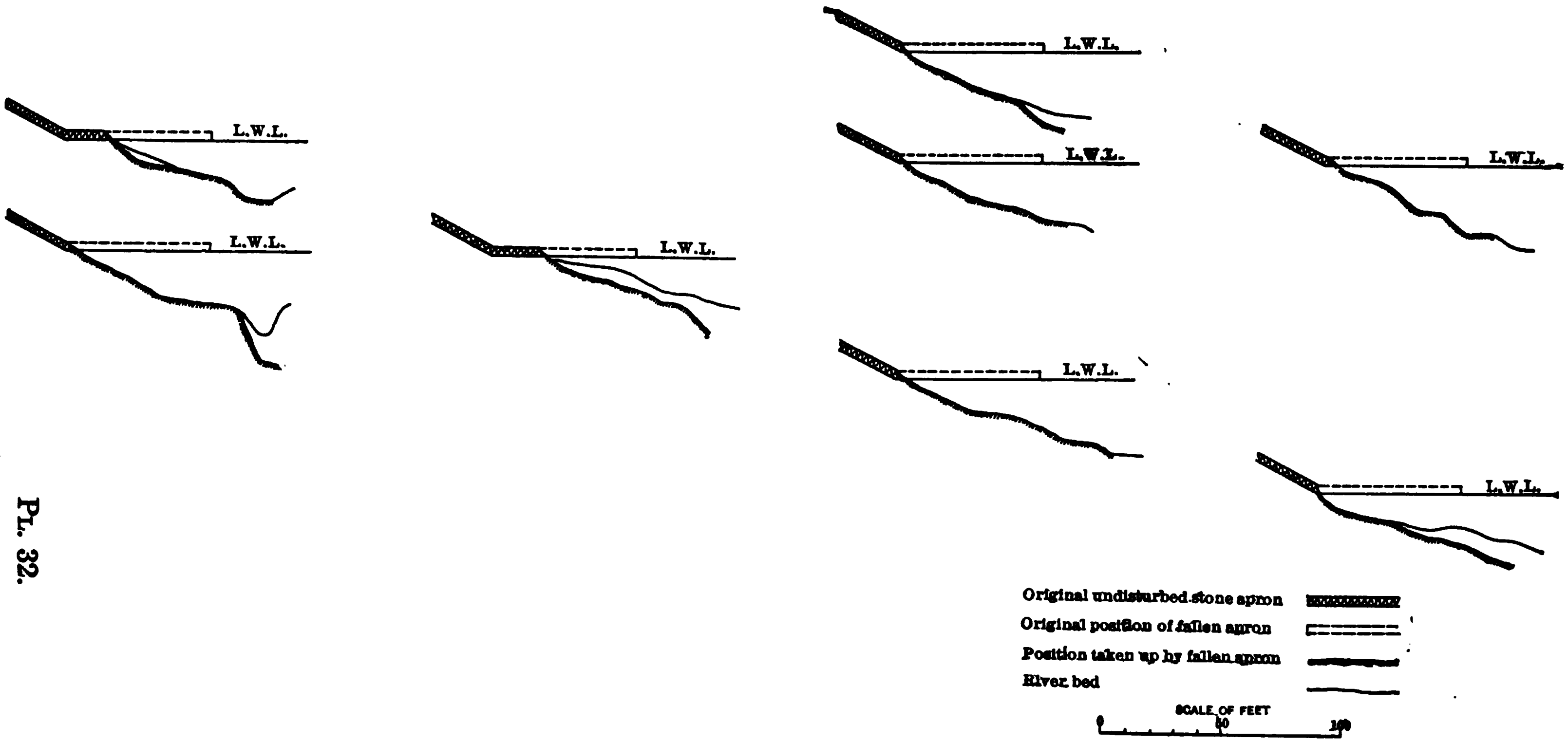
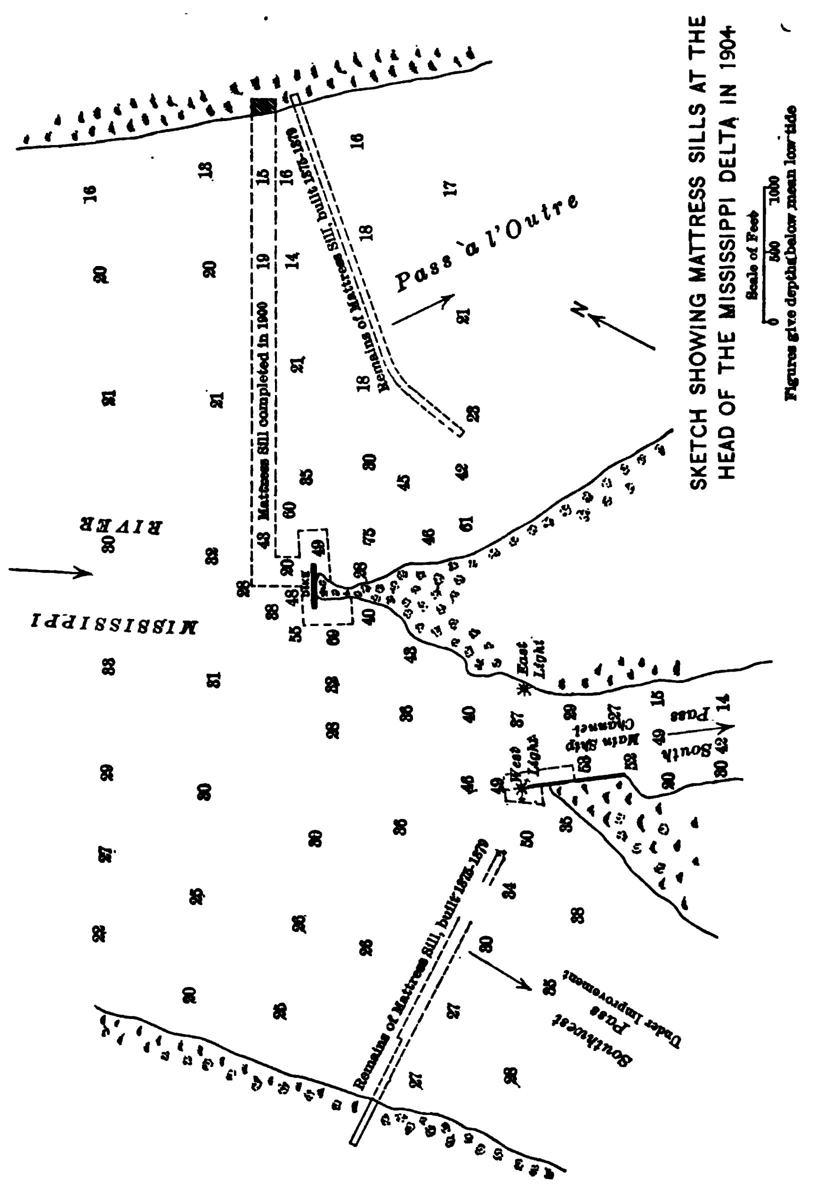



FIG. 4-DIMENSIONS OF GUIDE BANK APRONS.

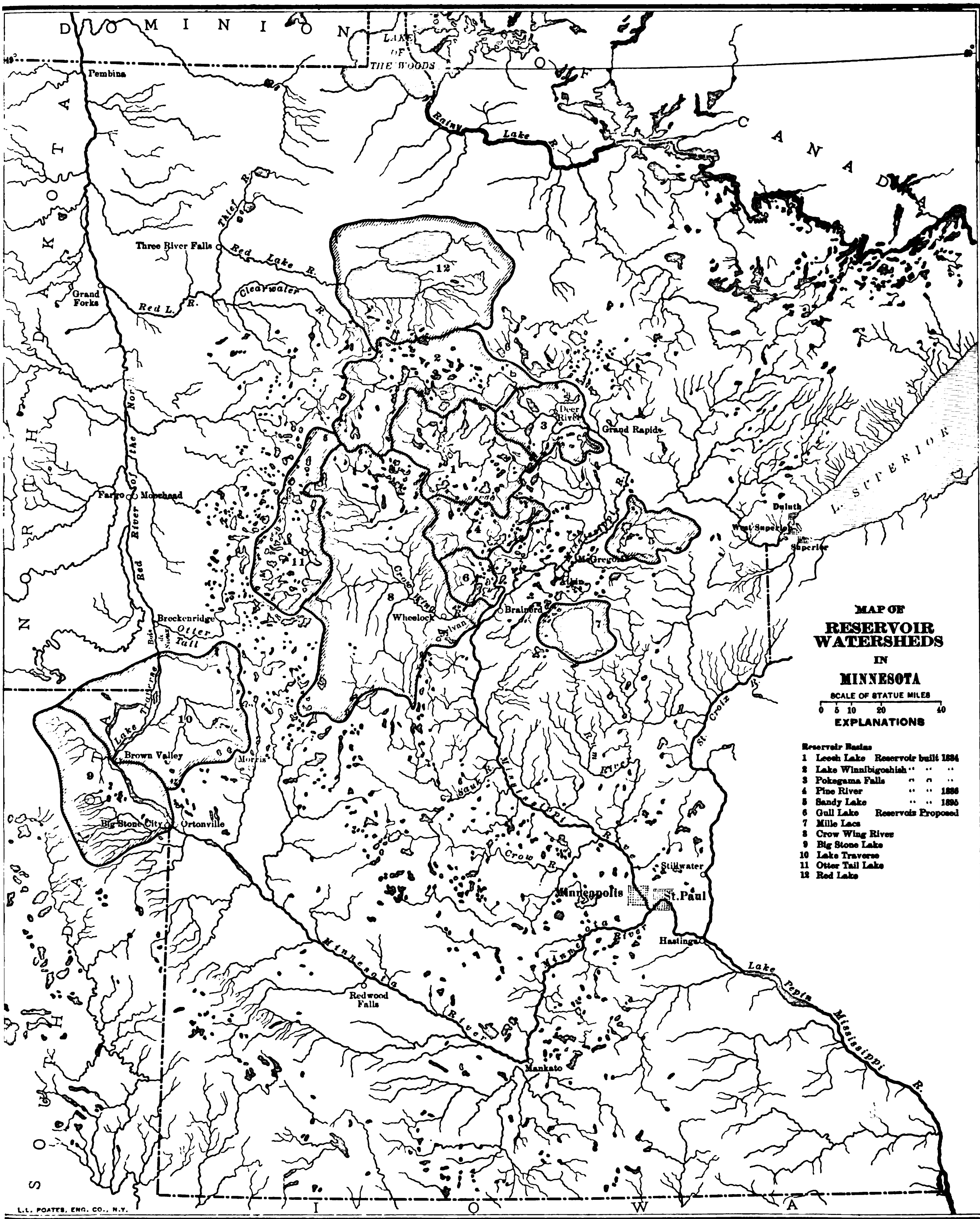
POSITIONS OF APRONS AS FOUND BY SOUNDINGS TAKEN ON THE SANKOS & GANGADHAR RIVERS, INDIA.




Pl. 32.
(Reference, p. 233.)




Pl. 33.
(Reference, p. 239.)




PL. 34.

(Reference, p. 304.)

PLATE 35

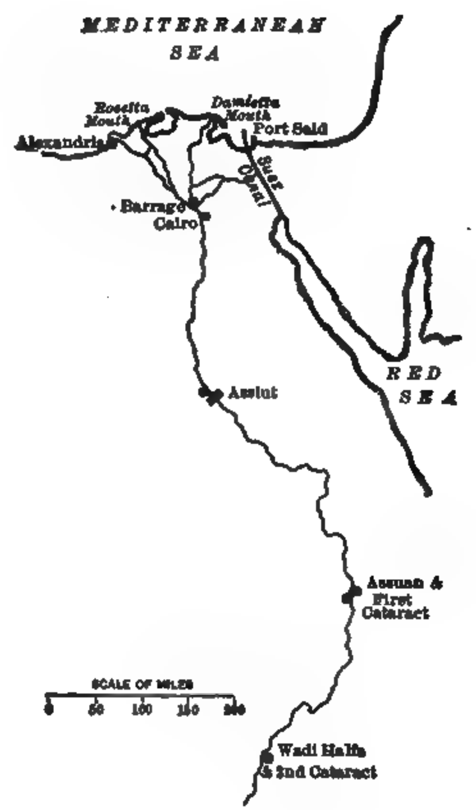



FIG. a-MAP OF THE NILE, NORTH OF THE SECOND CATARACT


FIG. b-GENERAL PLAN OF THE ASSUAN DAM, WITH RESERVOIR FILLED

those of the
See page 307.

Fig. c-SECTION OF SOLID DAM

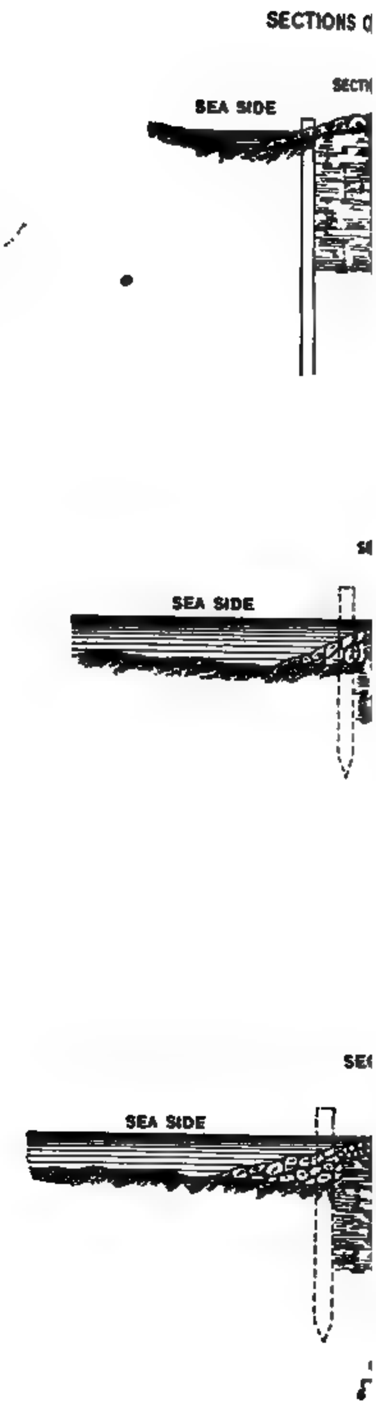
Fig. d-SECTION OF SLUICE


LOCATION AND SECTIONS OF THE ASSUAN DAM, RIVER NILE.

PL. 35.
(Reference, p. 305.)

PLATE 36

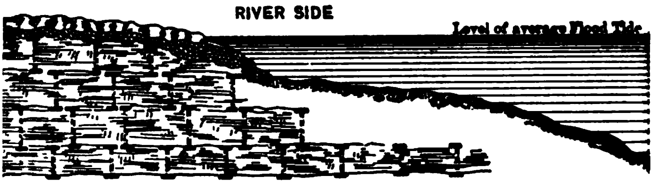
Fig. 1.


IMPROVEMENT

JETTIES AT SOUTH PASS

Fig. 2
OF INNER PORTION




B

Fig. 3
ION AT 9000 FEET

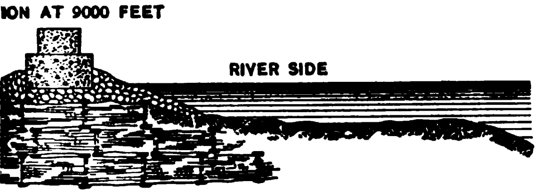
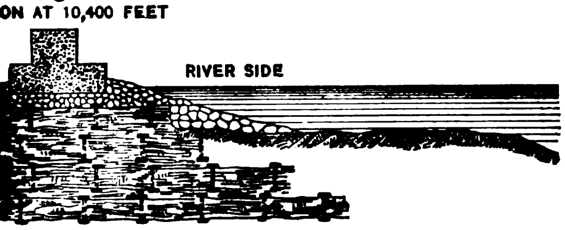


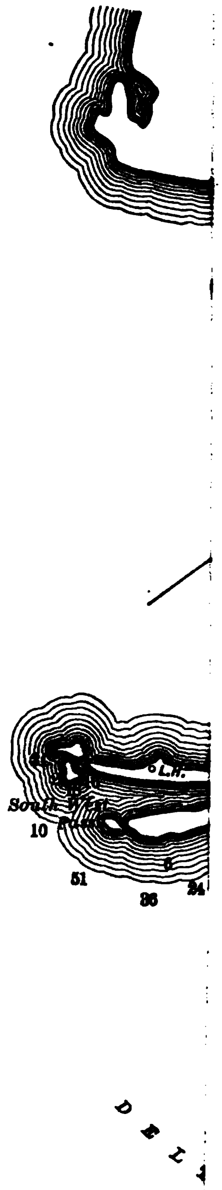

Fig. 4
ON AT 10,400 FEET




LE OF FEET FOR FIGS. 2, 3 AND 4.

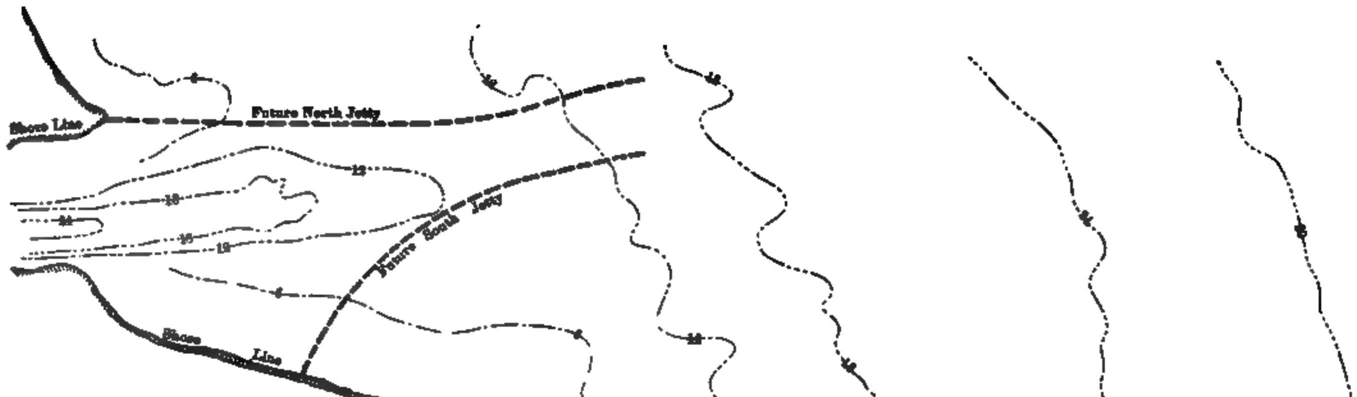
2 1 0 5 10 15

## OF SOUTH PASS, MISSISSIPPI RIVER

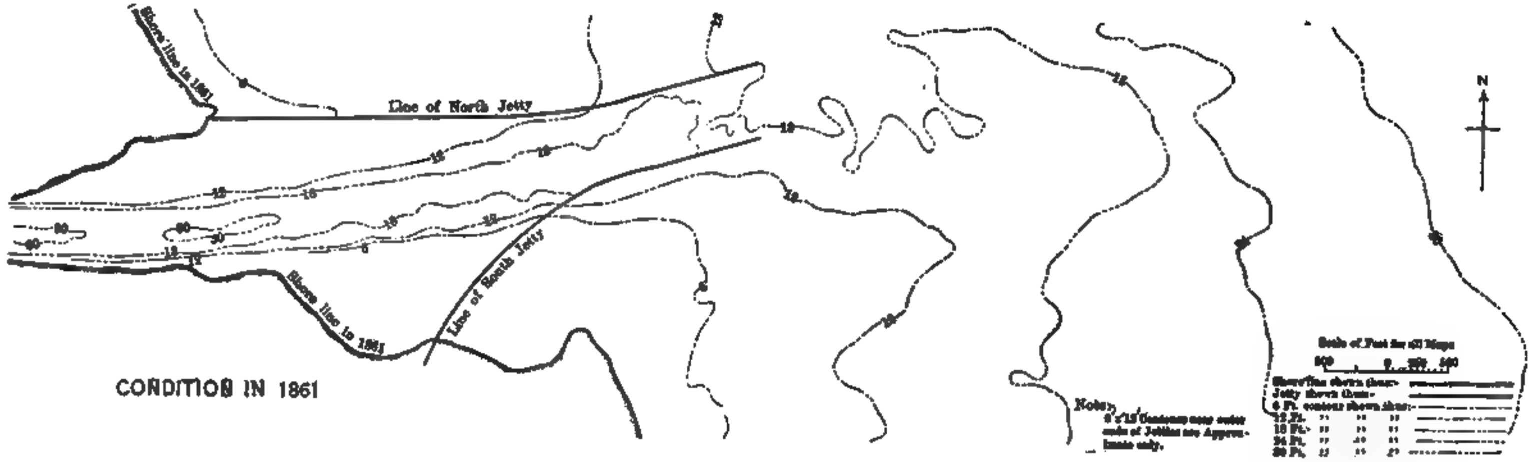


L.H.
South West
10
6
51
36
24

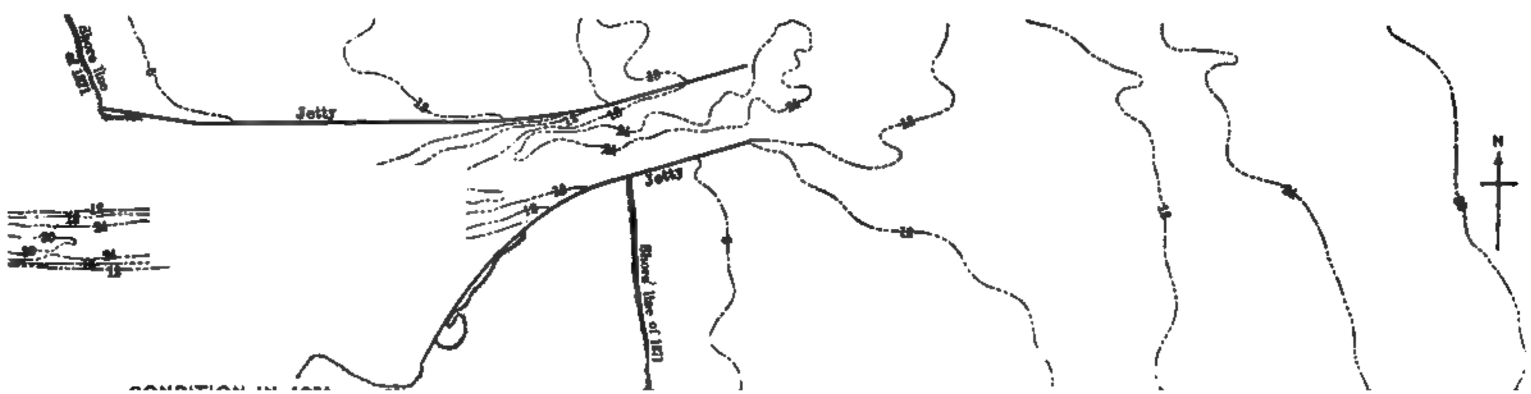
**PLATE 39**

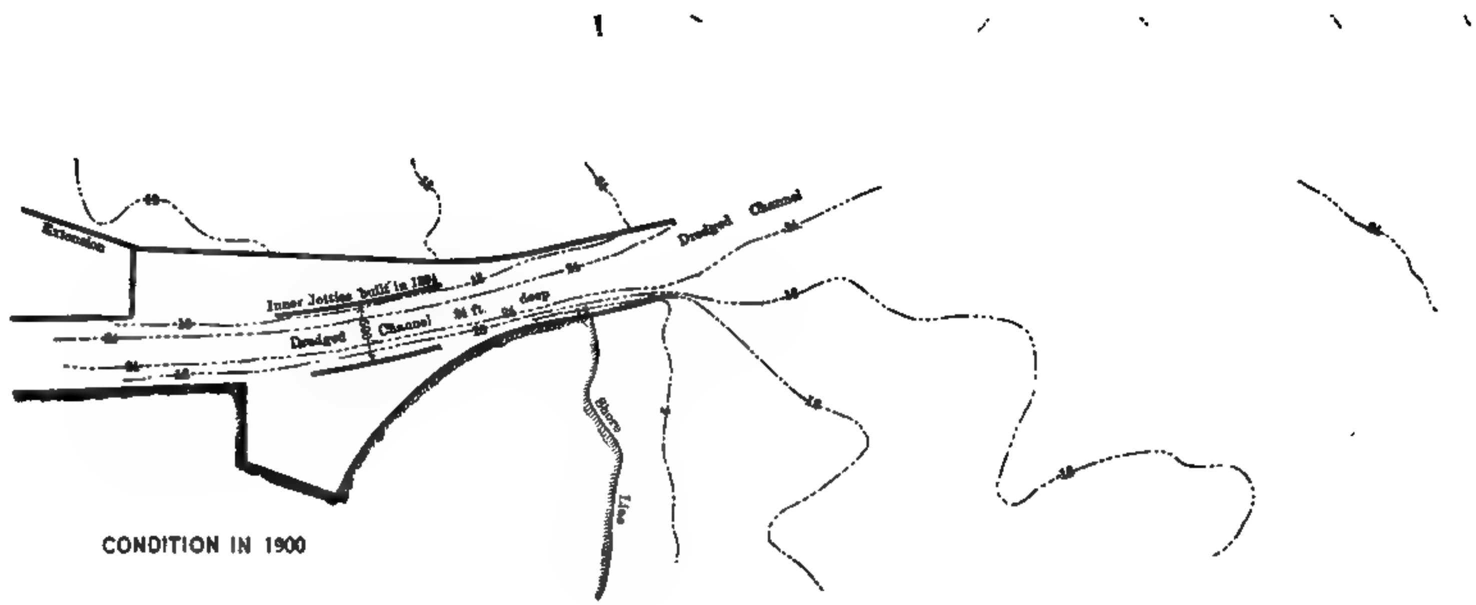


CONDITION IN 1857



CONDITION IN 1861



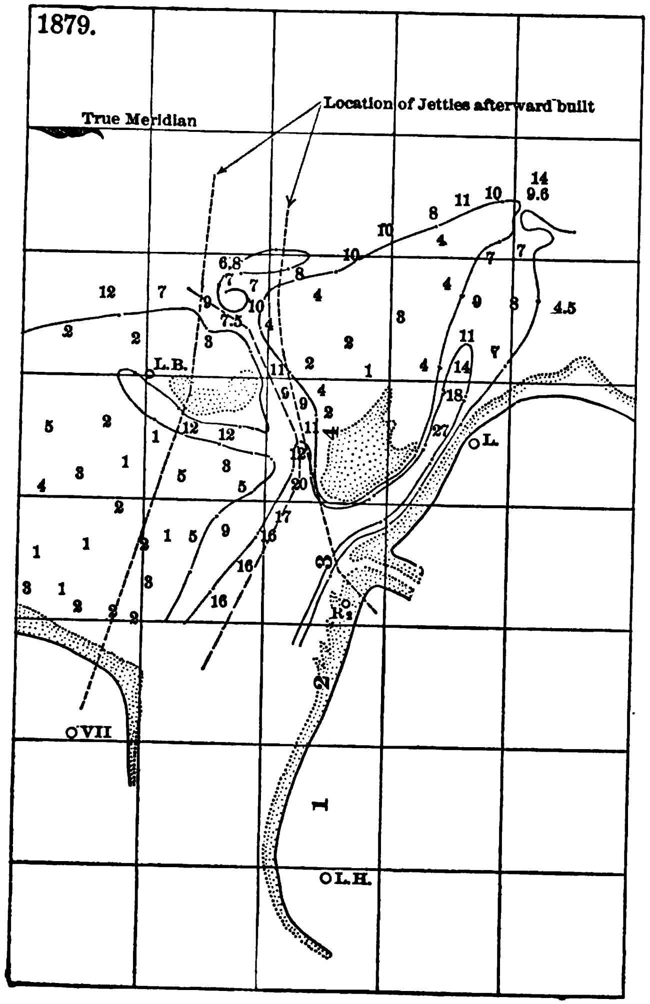


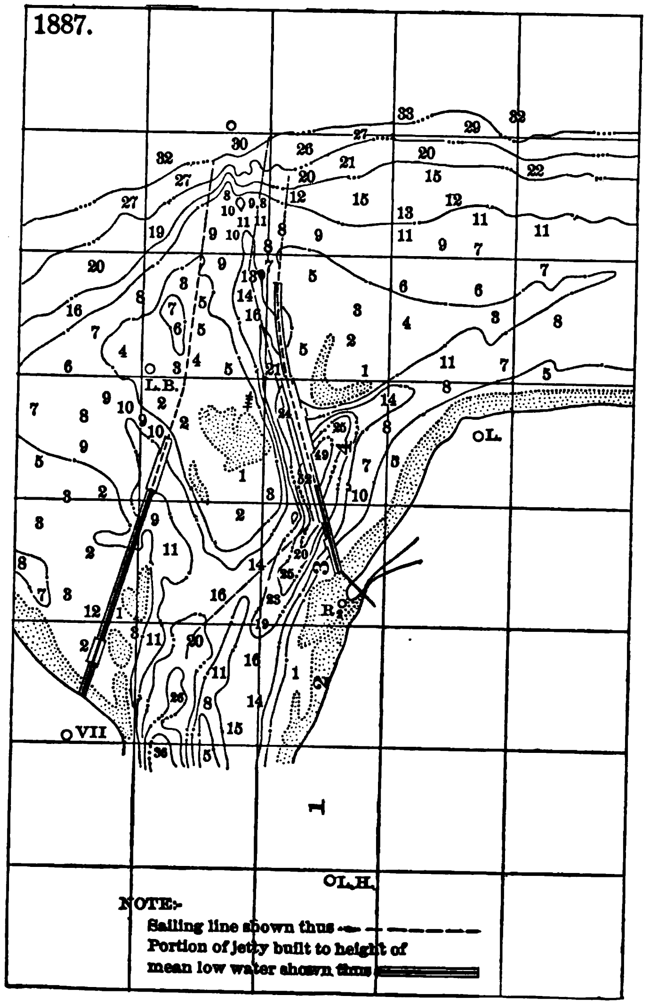

CONDITION IN 1900

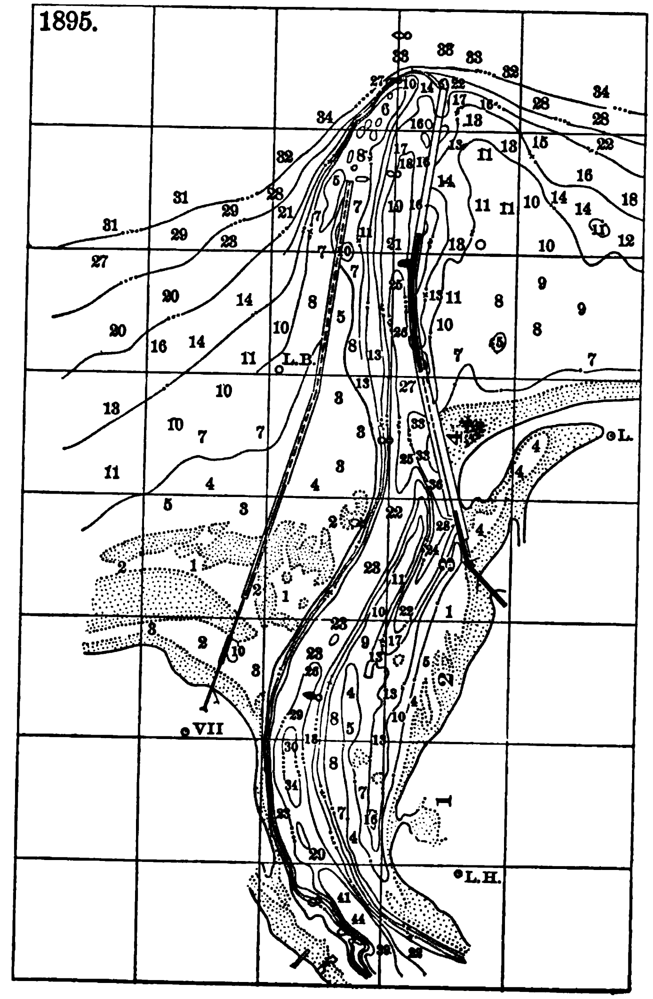

IMPROVEMENT OF THE SULINA MOUTH OF THE DANUBE.

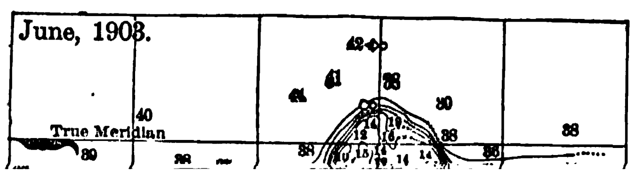
PL. 39.
(Reference, p. 327.)

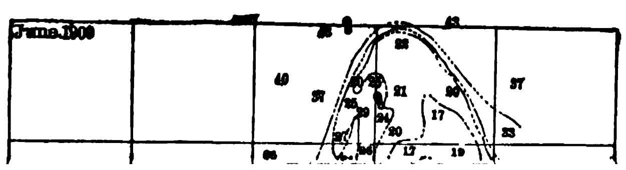
PLATE 40

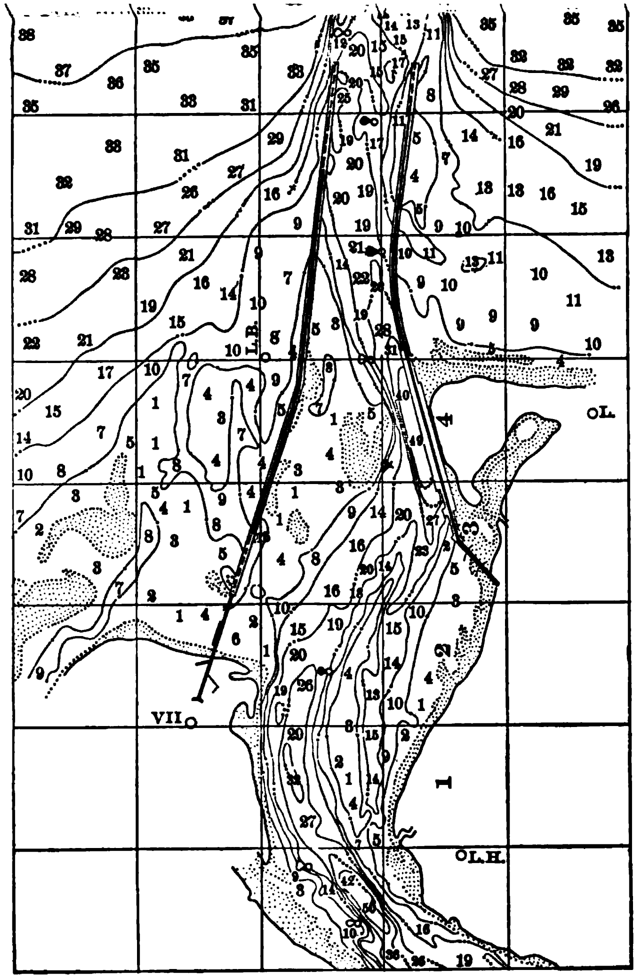

1879.
True Meridian
Location of Jetties afterward built
L.B.
L.
VII
L.H.

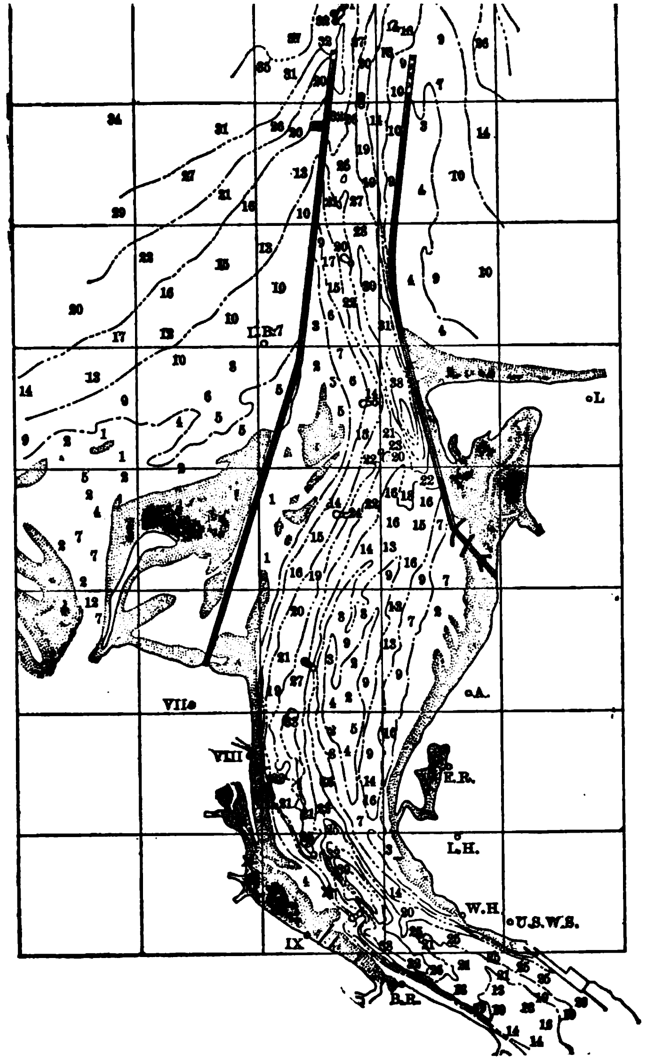

1887.
L.B.
L.
VII
L.H.
NOTE:-
Sailing line shown thus
Portion of jetty built to height of
mean low water shown thus


1895.
L.B.
L.
VII
L.H.


June, 1903.
True Meridian






NOTE:-
Crest of Jetties below Mean Low Water
" " " at " " "
" " " between M.L.W. and M.H.W.
" " " at or above Mean High Water

IMPROVEMENT OF THE MOUTH OF THE ST. JOHN'S RIVER, FLORIDA.

Scale of Feet
0 1000 2000 3000 4000 5000

**Pl. 40.**

(Reference, p. 328.)

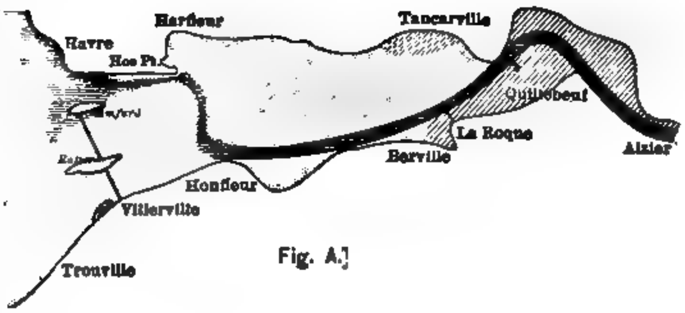



Fig. A.

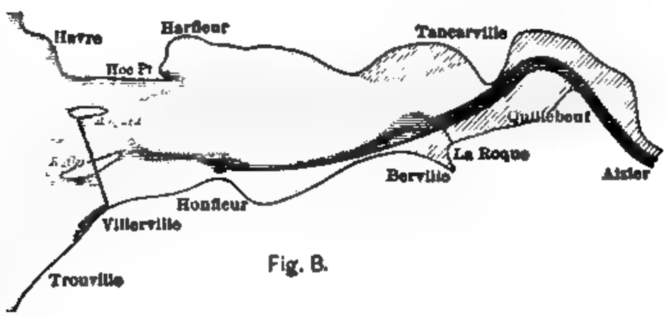



Fig. B.

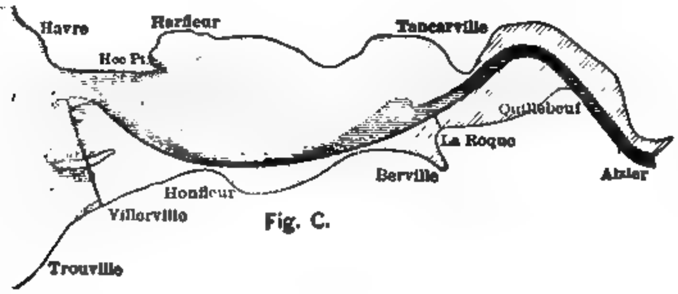



Fig. C.

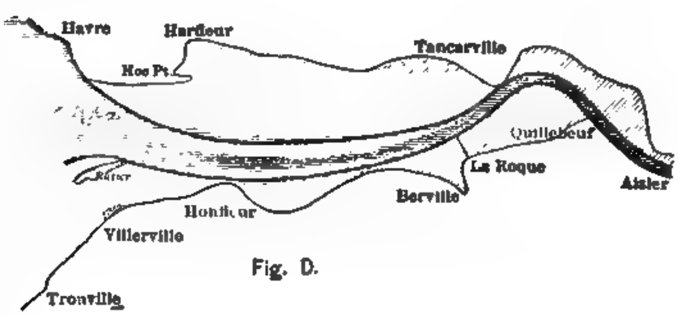
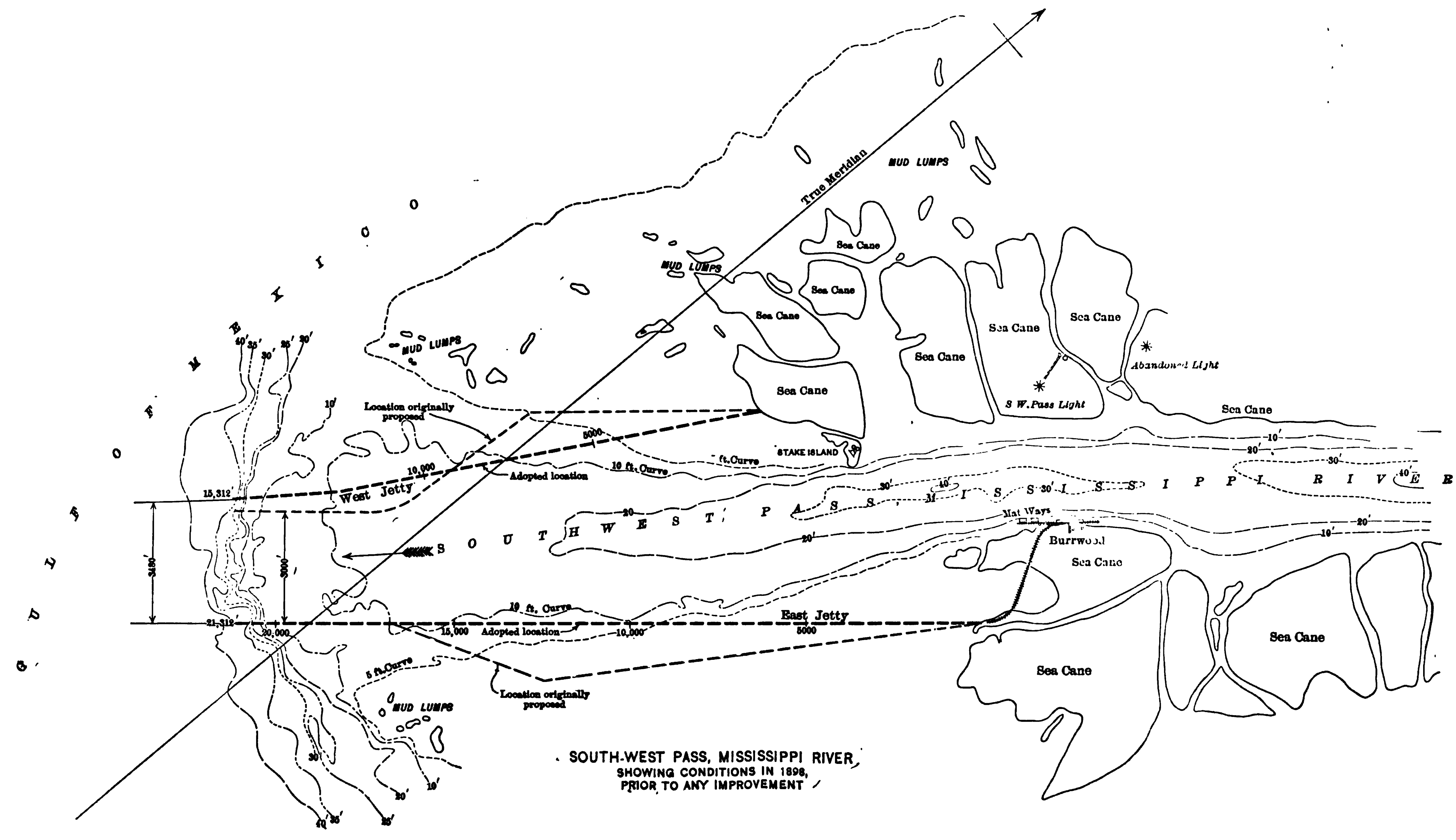



Fig. D.

RESULTS OF EXPERIMENTS WITH A MODEL OF THE ESTUARY OF THE RIVER SEINE.

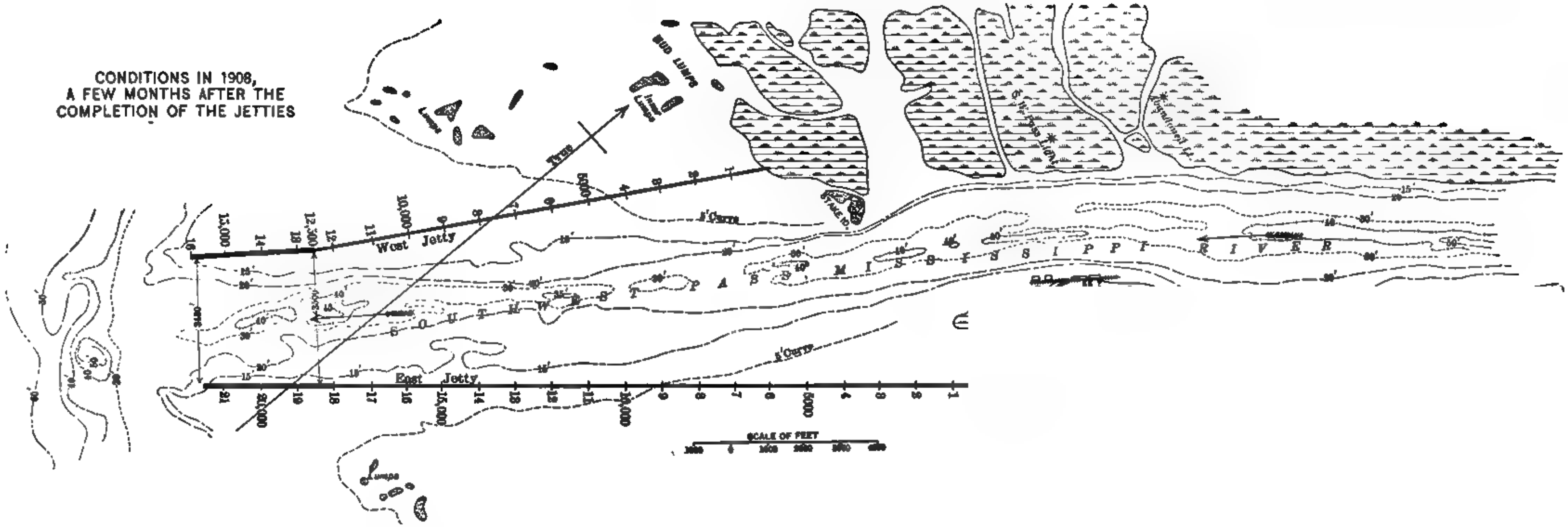
**Pl. 41.**
**(Reference, pp. 320–323.)**

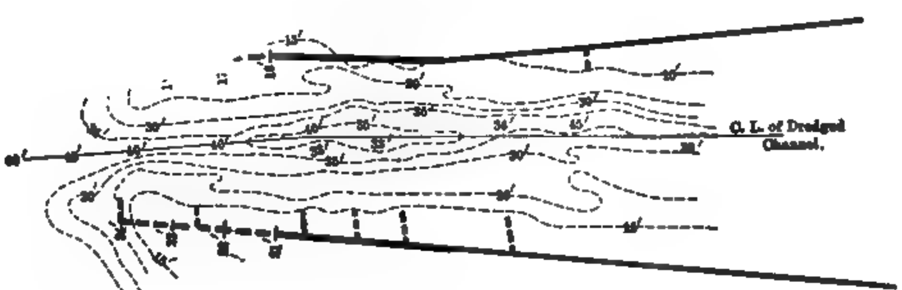
PLATE 42




SOUTH-WEST PASS, MISSISSIPPI RIVER,
SHOWING CONDITIONS IN 1898,
PRIOR TO ANY IMPROVEMENT

CONDITIONS IN 1908,
A FEW MONTHS AFTER THE
COMPLETION OF THE JETTIES



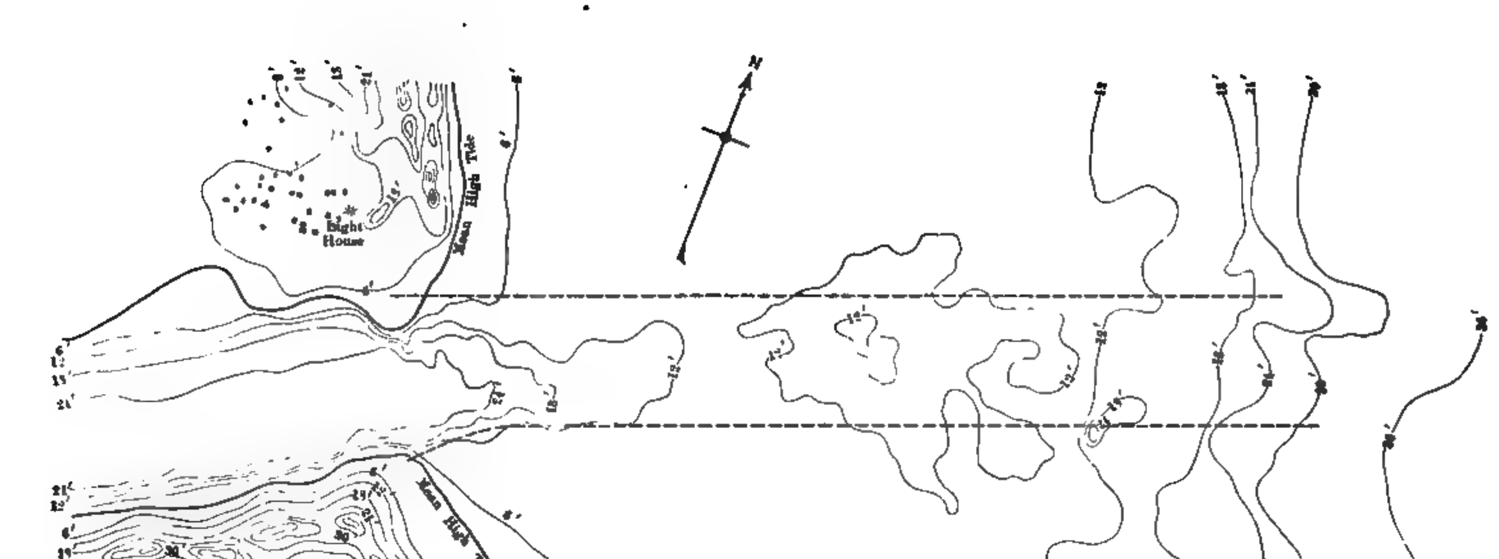

CONDITIONS IN 1911.
(Dotted portions show spur-dikes & jetty extensions completed or under construction in 1911.)

IMPROVEMENT OF THE SOUTHWEST PASS, MISSISSIPPI RIVER.

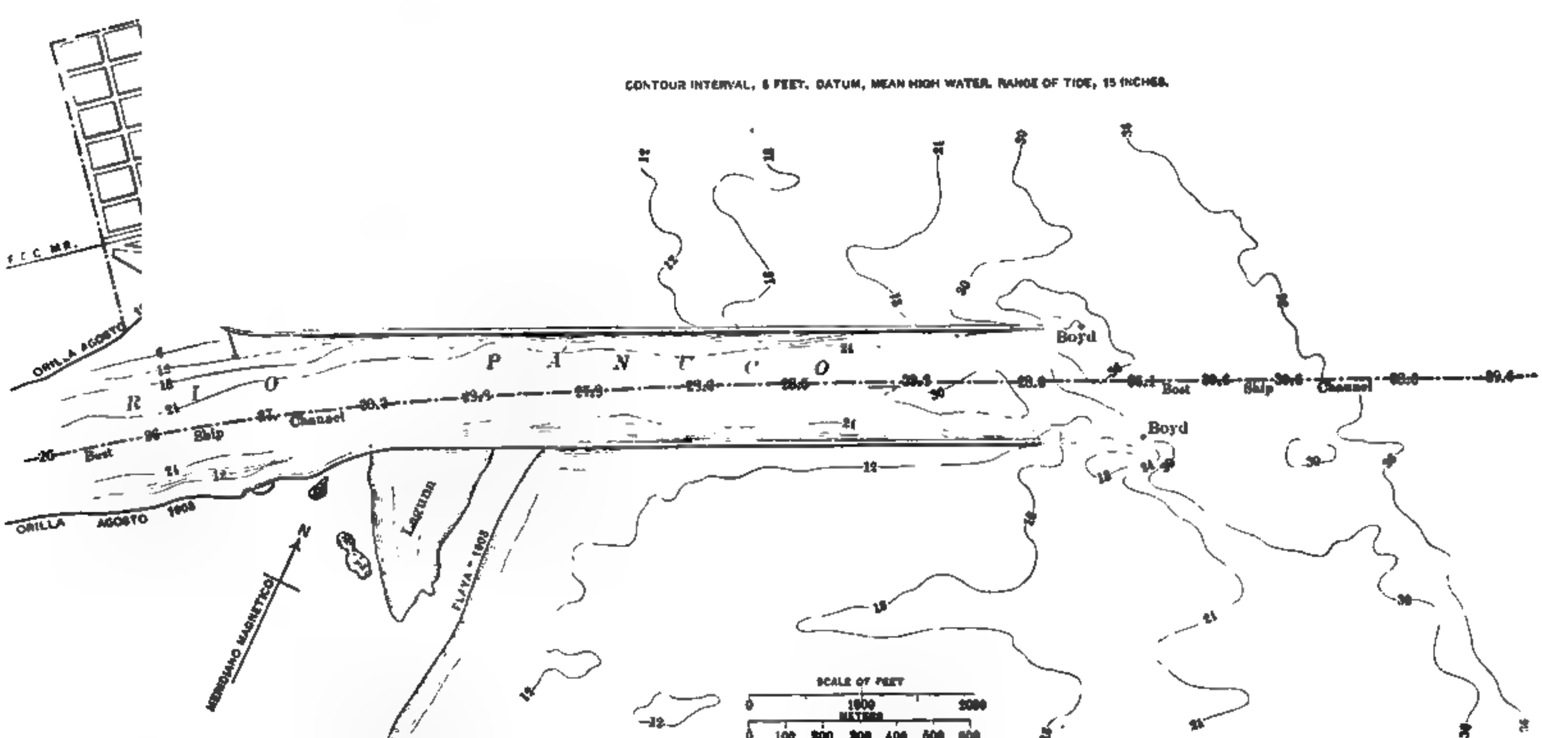
PL. 42.
(Reference, p. 324.)

PLATE 43

(Referen




CONDITION IN JUNE 1889 OF THE MOUTH OF THE RIVER PANUCO BEFORE CONSTRUCTION OF JETTIES.




CONDITION IN NOVEMBER, 1903, OF THE MOUTH OF THE RIVER PÁNUCO 11 YEARS AFTER THE CONSTRUCTION OF THE JETTIES.

RELIEF MODEL, SHOWING CONDITIONS BEFORE AND AFTER IMPROVEMENT.

IMPROVEMENT OF THE PANUCO RIVER, PORT OF TAMPICO, MEXICO.

PL. 44.

(Reference, p. 329.)

PLATE 45

ƒ S. Re 2 vals Soc
t. Es ret